Appendix X

U.S. NAVY
SPECIFICATIONS AND GUIDELINES
For

MARINE CONCRETE REPAIR

ABSTRACT

The U.S. Navy is continually looking for the best methods and materials to repair its reinforced concrete waterfront structures. The goal is to identify methods and materials that provide at least 15 years of service. Corrosion of the steel reinforcement is the paramount failure mechanism for Navy waterfront facilities. Sometimes the repair area is so extensive that "patching" the concrete is not practical and replacement is the favored alternative. Therefore, this document addresses new construction criteria in addition to repair methods and materials. Corrosion activity manifests itself as cracks, spalling, delamination, and eventually the reduction of structural capacity and operational readiness. Corrosion mitigation methods and materials are emphasized through the use of low shrinkage cementitious repair materials, proper concrete cover, and proper surface preparation and placement techniques that result in durable repairs. Epoxy-coated rebar is recommended for new construction because it provides supplemental corrosion protection. Discussions, guidelines, specifications, and illustrative details are provided for inspection, new construction, and repairs above and below the waterline.

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PART A

CONDITION ASSESSMENT AND INSPECTION

SUMMARY

The average age of Navy waterfront structures is about 50 years and they typically exhibit deterioration due to rebar corrosion, cracking, and carbonation. Rebar corrosion is usually very active in the splash zone and at curbs, joints, and where concrete cover over the rebar is insufficient. Prior to repairs being made, a comprehensive inspection of the concrete is necessary to delineate the amount of concrete needing repair. Guidelines for condition assessment are contained in various industry standards.[1,2]

During the removal of the damaged concrete and while the contractor is "chasing" the corroded rebar to a point where it is not significantly corroded, it is typical to uncover areas that need to be repaired that were not identified in the initial inspection. Non-destructive inspection techniques and tools are not always adequate to predict the extent of repairs that need to be made. Because of the frequent occurrence of "going over budget" during repairs, it is common for planners and estimators to increase the estimated repair amounts when preparing the repair budget. Contingentency factors of two to four times are typically used by some planners. Even then, many repair projects go over budget.

INSPECTION TOOLS

The degree of deterioration is often much more extensive than is first apparent. Many tools are available to collect data to make repair estimates, the usual tools are summarized below:

Delaminations are usually detected by using a hammer or chain.

Powder samples taken from the top and bottom deck are used to measure the degree of chloride contamination at the depth of the rebar. Values that exceed 1.5 pounds per cubic yard are at the threshold at which rebar corrosion may occur if sufficient moisture and oxygen are present.

A pachometer (rebar locator) may be used to measure the depth of concrete cover over the rebar. Rebar that has less than 1.5 inches of cover is very likely to be corroded or will corrode.

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[1] Military Handbook Maintenance of Waterfront Facilities Sept 1997 MO-104
[2] Guide for Making a Condition Survey of Concrete in Service ACI 201.1R-84

A Schmidt Hammer may be used to approximate the concrete's compressive strength according to ASTM C 805 (Standard Test for Rebound Number of Hardened Concrete). Variations in surface strength may indicate areas of concrete that that are soft from carbonation or delamination.

A petrographic analysis can be very useful to approximate the water-to-cement (w/c) ratio, quantify the cement paste-to-aggregate bond, and to identify other failure mechanisms such as alkali silica reaction and the formation of ettringnite. Higher w/c ratios are generally associated with greater permeability.

A portable adhesion tester may be used to determine the concrete's surface tensile strength according to ASTM D 4541 (Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers). Concrete in good condition will exhibit tensile strengths between 300 psi to 500 psi. Surface condition is important information if coatings or membranes are to be applied as part of the repair. If coatings are used, a vapor emission test is necessary and the results are expressed in pounds of vapor per 1,000 square feet. Coating selection is dependent on vapor transmission.

PART B

NEW CONSTRUCTION
(REPAIR BY REPLACEMENT)

Repairs to marine concrete facilities are typically required because of insufficient cover to the reinforcement or inadequate concrete quality. In other cases, the structure may have been damaged by ship impact or a seismic event. When repairs are very extensive, it may be more economical to replace all or part of the structure. Part B addresses the scenario of "repair by replacement."

CHAPTER 1

CONCRETE DURABILITY

CONCRETE DURABILITY

INTRODUCTION

This chapter has been extracted from an introduction to the subject of durability of marine concrete structures[1]. It addresses the general deteriorating mechanisms that may occur in concrete structures and fundamental guidelines for specifying durable materials for reinforced concrete in a marine environment.

All materials are vulnerable to destructive actions. The destruction can be slow or rapid, depending on the material and the surrounding environment. Materials that are very durable in one environment may deteriorate rapidly in another environment. The ability of materials to withstand destructive actions is expressed by the term durability.

The durability of one specific material naturally varies with the type of attack. Steel within concrete can be destroyed by electrochemical attack or corrosion, for which the concrete itself is completely unsusceptible. The concrete, however, can be affected by other destructive forces of chemical and physical origin, such as deterioration from different aggressive chemical substances, deterioration by means of frost action, deterioration by abrasion, and so on.

Concrete is a porous material, porosity being the presupposition for almost every form of deteriorating forces. Concrete may also be vulnerable to cracking due to thermal movements, shrinkage, and moisture movement. The pore system and small cracks make it possible for different substances to enter the interior of the reinforced concrete and attack the different elements that constitute the material. Water, oxygen, and chlorides can reach the steel reinforcement and destroy it by means of corrosion. Water that fills up the pore system and small cracks can freeze, expand, and deteriorate the concrete. The permeability of the concrete, therefore, is a main parameter governing the durability of concrete.

STEEL REINFORCEMENT CORROSION

The most serious and probably most frequent type of deterioration of marine concrete structures in general is reinforcement corrosion. In a young concrete structure, the steel reinforcement does not corrode because of the alkaline nature of the surrounding concrete. The steel is said to be in a passive state. The adequacy of the alkaline protection is dependent upon the thickness of the concrete cover to the reinforcement, the quality of the concrete, the details of the geometry of the structure, the degree of chlorides in the concrete constituent materials, and external sources.[2] [3] However, over a period of time the protecting environment may disintegrate and the passivity destroyed. This may happen by:

Carbonation of the concrete cover caused by ingress of carbon dioxide

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[1] Draft report on "Durability of offshore concrete structures," prepared by Aker Maritime, Lars Bjerkeli, for the Office of Naval Research, Feb 1999.

[2] ACI 201.2R-92: Guide to durable concrete.

[3] ACI 222R-96: Corrosion of metals in concrete.

Penetration of chloride ions to the reinforcement

Sound marine concrete has a very low permeability. A permeability coefficient of less than 10^{-12} m/s measured on drilled samples is required according to the Norwegian Petroleum Directorate[4]. Carbon dioxide penetration in concrete with this low permeability is so slow that carbonation is not an issue for marine concrete.

The required low permeability and the minimum concrete cover requirements according to Norwegian Standard NS 3473[5] are the main efforts implemented to attain a chloride ion level adjacent to the reinforcement that is low enough not to break the reinforcement passivity. Even if the steel passivity is broken, corrosion requires an electrolytic cell in which oxygen must penetrate the concrete cover and reach the steel. The low permeability of the concrete cover and the low diffusion rate of oxygen from the seawater significantly reduce the possibility of oxygen reaching the reinforcement.

If the concrete is cracked, a more rapid penetration of chlorides to the steel may take place. The effort to reduce this deteriorating effect to an acceptable level is limitation of the crack widths. The most stringent crack width limitations apply to the splash zone and the atmospheric zone. These zones are more or less continuously subjected to wetting and drying and there is enough oxygen to sustain the corrosion process. In addition, frost action may occasionally occur.

A more relaxed crack width limitation applies to the permanently submerged zone. In this zone, cracks that permit water ingress tend to close themselves by additional hydration of the cementing materials, autogenous healing, and deposition of filling materials. Leaching of calcium hydroxide and other soluble substances from the cement paste fill up the cracks and may close them completely. Deposition of materials such as argonite and brucite resulting from reactions of the seawater and the cement will also seal cracks.

An extensive introduction to the influence of crack widths and self-healing of cracks on durability may be found in Jakobsen, et al.[6]

Deterioration of concrete due to corrosion may result in significant cracking because the products of corrosion (rust) occupy a greater volume than the steel and exert substantial stresses on the surrounding concrete. The outward manifestations of rusting include staining, cracking, and spalling of the concrete.

CORROSION PROTECTIVE MEASURES

The concrete mix design, sufficient cover to the reinforcement, and limitation of crack widths have already been mentioned as important factors to prevent reinforcement corrosion. These factors are introduced as requirements in the design basis.

Experience from pouring of concrete in full-scale production proves that a theoretically durable concrete mix design, tested and qualified by laboratory tests, may be difficult to apply in "real life" construction. This may be because of the applied production technique and equipment or due to high reinforcement densities, weather conditions, and or some other reason.

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[4] Norwegian Petroleum Directorate: Regulations for loadbearing structures in the petroleum activities. 1992

[5] Norwegian Standard NS 3473. "Concrete Structures, Design Rules", 4th edition 1992.

[6] S.Jakobsen, J,Marchand and B.Gerard: Concrete cracks I: Durability and self healing - A review. Proceedings of the second international conference on concrete under severe conditions, CONSEC –98, Tromsø, Norway

To avoid these potential problems, trial mixes should be qualified under full-scale production situations. Efforts to obtain the most optimal curing conditions must also be tested out prior to being applied in production. Non-structural cracks, i.e., cracks that are a result of unfavorable curing and hardening conditions rather than tensile stresses in the concrete caused by external loads, are often a larger concern than structural cracks. According to Mehta,[7] thermal shrinkage and drying shrinkage is the primary cause of cracks in many new concrete structures. These cracks are related to high early-strength concrete mixes that typically have a relatively high content of fine ground cement. In addition, the cement often has a relatively high sulphate and alkali content. These types of cracks are not controlled by the amount of minimum reinforcement.

Preventing the chlorides from entering the concrete is an additional effort that may be applied to reduce the potential corrosion problem. An extensive list of methods for controlling and monitoring corrosion may be found in ACI 22R-96.[8]

FREEZE THAW CONSIDERATIONS

The mechanism for freezing and thawing marine concrete is the same as for onshore concrete. It consists of two parts, one related to the material ("the strength") and one to the environment ("the load").

The material-related part is concerned with how much water within the concrete is sufficient to cause frost damage. This material property is called the critical degree of water saturation S_{cr}.

The environment-related part is concerned with how much water is present within the concrete. This part is called the actual degree of water saturation S_{act}. For frost damage to occur, S_{act} has to be as high as S_{cr}. or higher.

The following recommendations apply to concrete that will be exposed to a combination of moisture and cyclic freezing.[9]

Design of the structure to minimize exposure to moisture
Low water–cement ratio.
Appropriate air entrainment
Quality materials
Adequate curing before first freezing cycle
Special attention to construction practices

SULPHATE ATTACK

Sulphate ions in the seawater can attack the calcium hydroxide in the cement paste, the final reaction product being ettringite. As ettringite is very voluminous, the result can be a volume increase with a subsequent disruption or softening of the concrete.[8] The concentration of

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[7] P.K.Mehta: Durability – Critical issues for the future. Concrete International, July 1997
[8] ACI 222R-96: Corrosion of metals in concrete.
[9] ACI 201.2R-92: Guide to durable concrete.

sulphate ion in seawater can be increased to high levels by capillary action and evaporation under extreme climatic conditions.

Low permeability, low water–cement ratio (w/c < 0.4), and a low C_3A content in the cement are the key factors to obtain acceptable sulphate resistance of concrete.

ALKALI-SILICA

Reactive siliceous minerals in the aggregates can be attacked by alkaline hydroxides in the pore water, the hydroxides derived mainly from the oxides of sodium and potassium in the cement paste. The reaction product is an alkali-silicate gel. The gel can absorb water resulting in an increase of volume and internal pressure, which eventually may cause cracking of the concrete. At low temperature, the reactions may become dormant.

The expansion of the gel is dangerous only for certain combinations of reactive aggregates, alkaline hydroxides, and water in the concrete. If the content of alkaline hydroxides is low (the equivalent amount of sodium dioxide should not be higher than 0.6% according to ASTM), the reaction is small and of no significance. Also, if the content of reactive silica aggregates is low or high, or the particle size is very small or very big, the reaction is small and of no significance.

In general, the reactivity of the aggregate should be tested before being recommended for use. Outline of test methods, criteria for judging reactivity, and recommended procedures to be used with alkali-reactive aggregates may be found in footnote 8.

ALKALI-CARBONATE

Some dolomitic limestone aggregates can react with the alkaline hydroxides in the pore water. The result is an expansion similar to that occurring as a result of the alkali-silica reaction described above. The expansion causes a network of pattern or map cracks usually most strongly developed in areas of the structure where the concrete has a constantly renewable supply of moisture. The expansion can be very severe, and has led to extensive problems onshore in Canada and the U.S. Fortunately, reactive carbonate rocks are not very widespread and can usually be avoided.

An outline of test methods, criteria for judging reactivity, and recommended procedures to be used with alkali-reactive aggregates may be found in ACI 201.2R-92.[8]

SULPHUR REACTION

Sometimes aggregates contain sulphur compounds, usually as sulphates and sulphides, which can be the cause for deleterious expansions. Sulphides, together with water and oxygen, can produce reaction products similar to those which can be produced by sulphates. Careful testing and examination of the aggregates will usually indicate the presence of such reactive impurities and their use in concrete avoided.

LEACHING

Under special circumstances, submerged concrete may undergo some leaching of the more soluble substances (mainly calcium hydroxide) from the cement paste. The actual leaching is very small and probably takes place mainly in cracks, where it serves the useful purpose of helping to close the cracks.

WATER TIGHTNESS

The water permeability of uncracked concrete is governed mainly by the cement paste and the contact zone between the aggregates and the paste.

Concrete with a water/cement ratio below 0.45, which is the case for the North Sea marine structures, is for all practical purposes non-permeable with respect to transport of liquids through the section thickness. Tightness is therefore primarily a matter of avoiding through cracks that may arise during hardening of the concrete or during platform construction and operation. A minimum requirement for amount of ordinary reinforcement is equally important for the efficient distribution into many small cracks rather than a relatively few larger cracks. In addition, special attention has to be paid to construction joints.

CONSTRUCTION JOINTS

Potentially, construction joints can act as waterways through a concrete structure. To avoid this completely, it is necessary to have skilled labor and well-proven routines to treat the joints properly.

The routines vary quite a lot, depending on the situation. Typically, the concrete surfaces at all joints are thoroughly cleaned prior to placing adjoining concrete. Then, if possible, a layer or concrete with increased cement content and workability is placed against the joint, followed immediately by ordinary concrete, with vibration through the first layer.

If water tightness of a construction joint is required, procedures must be prepared to test the joint for leakage before construction work is completed. Observed leakage is repaired with epoxy injection. Precautions can be taken by pre-installation of epoxy injection tubes in the casting joint and injection of epoxy after hardening of the concrete.

CLIMATIC EFFECTS

The temperature will affect the rate of the deteriorating chemical processes involved, but the processes themselves will be basically the same. Other important factors that may influence the deteriorating processes are the oxygen content and the salinity of the seawater. Different experiences in various climatic zones are often related to quality control during construction, the local material sources applied, and the conditions during construction; not to the environment itself.

In Fookes, et al.[10] the climatic situation world-wide is divided into four types: (1) hot wet, (2) hot dry, (3) temperate, and (4) cool (which also includes freezing). Their characterization is shown in Figure 1, together with a world ocean salinity and temperature chart.

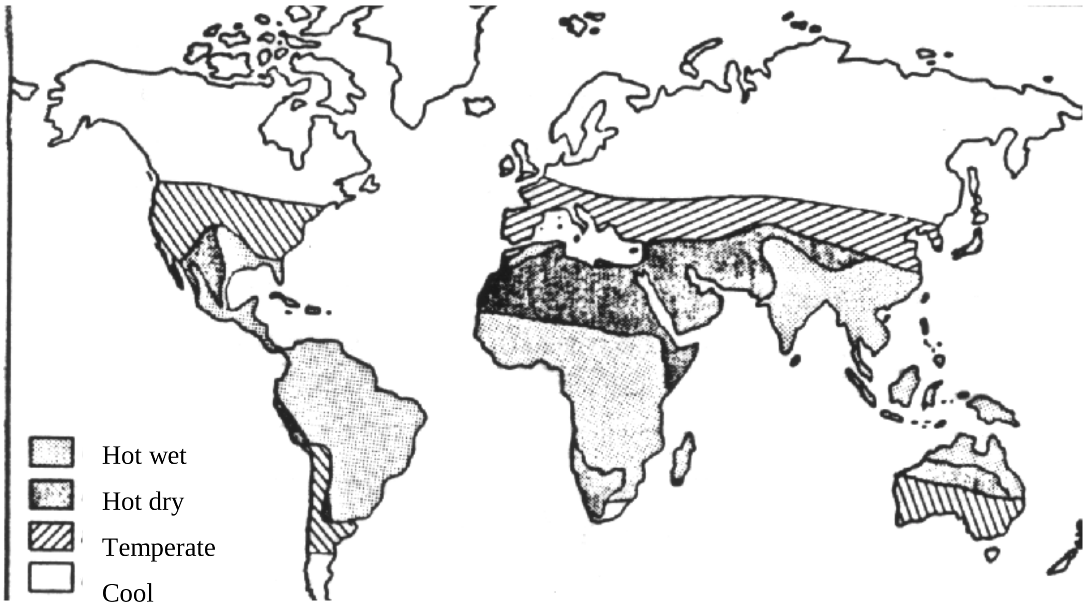

Figure 1. World climatic regions and world ocean and salinity and temperature chart [9]

HOT-WET REGIONS

According to ACI committee 305[11] the definition of hot weather is any combination of the following conditions: (a) high ambient temperature, (b) high concrete temperature, (c) low relative humidity, (d) high wind velocity, and (e) solar radiation. A comprehensive presentation of recommended practice and precautions for hot weather concreting may be found in RILEM TC 94CHC[12] and footnote 10.

SERVICE LIFE CALCULATIONS FOR MARINE CONCRETE STRUCTURES

Analytical models have been developed to be able to calculate the service life of marine concrete structures. The basis for the models is that reinforcement corrosion due to chloride ingress is the main deteriorating mechanism. The traditional assumption has been that chloride ingress into concrete obeys Fick's second law of diffusion for a semi-finite medium with constant exposure, and that there is a critical value of the chloride content in the concrete, $C = C_{cr}$, leading

[10] P.G.Fookes, J.D.Simm and J.H.Barr: Marine concrete performance in different climatic environments, Marine concrete '86 London Sept.1986.
[11] ACI Committee 305; Hot weather concreting. ACI chapter 305 R-91 American Concrete Institute
[12] RILEM TC 94CHC (1993): Concrete in hot weather environments. Draft "Part I: Influence of the environment on reinforced concrete durability. Part II: design approach for durability".

to the corrosion of the steel. Typical parameters in such models are chloride background content, chloride content on the exposed surface, exposure time, penetration depth, and diffusion coefficient. Recent research has shown that the diffusion resistance improves over time and that this has to be taken into account in analytical models, i.e., the diffusion coefficient is time dependent.

SUMMARY

The key requirements for new marine concrete structures are low initial cost, low maintenance costs, and a long service life. The main requirements for production of durable marine concrete in all environments are summarized as follows:

Make concrete as dense as possible
Use fairly rich mixes, i.e., 350 kg to 450 kg of cement per m^3 of concrete.
Use appropriate cover to the reinforcement.
Use sulphate-resisting cement and non-reactive aggregates.
Keep chloride content at a minimum in the concrete mix
Consider the use of air-entraining agent to improve frost resistance if required
Avoid thin sections and complicated geometry for properly pouring of the concrete.
Apply the highest standards of workmanship and supervision
Consider the effects of ambient temperature and environment during preparation of procedures for casting and curing.

CHAPTER 2

MIX DESIGN

MIX DESIGN

INTRODUCTION

This chapter is an extract[1] that describes some very significant developments in concrete technology and site practices have taken place over the past decades enabling improved concrete qualities to be produced on site. By means of selected constituent materials and mix proportions, high strength or high performance concrete can be produced to uniform and predictable levels of quality which will ensure long maintenance free service life under rough and hostile conditions.

Concrete mix design is an interactive process where selected constituent materials are combined to meet the technical requirements of the design and, at the same time, meet the practical performance requirements of the chosen plant and construction procedures on site. The design requirements apply primarily to the hardened concrete (strength, durability, ductility, density), whereas the site performance requirements apply to the fresh concrete (workability, pumpability, curing).

The process starts in the laboratory or with existing records with a view to identify suitable constituent materials. The requirements are laid down in relevant national and international Codes or Standards, supplemented with additional project-specific requirements. Tests may also be required to evaluate the mutual compatibility of individual materials.

The concrete strength and other mechanical properties are a function of the w/c, or water to binder, ratio. A low w/c ratio is required both for strength and durability reasons. To achieve this, while maintaining adequate workability and a moderate cement content, is often the key to a successful mix design. (The term w/c ratio is used throughout this report to define the ratio between water and total cementitious material, i.e., cement + silica fume + PFA or other pozzolans.)

Heat of hydration, or the temperatures generated during curing and hardening, sets up thermal stresses and may lead to undesirable cracking of the young concrete and impaired durability. The type and dosage of cement is the key factor in this respect and a low cement content consistent with the required strength and w/c ratio is desirable. In this respect, a combined binder of Portland cement, PFA, and/or silica fume may be an attractive option. Special curing measures may also be required to limit temperatures and temperature gradients in the early phase.

The target mean strength of the concrete required to achieve the stipulated characteristic value depends on the uniformity of production (coefficient of variation) which can be maintained. The efficiency of the Quality Assurance/Quality Control (QA/QC) system therefore also affects the mix design process.

The above demonstrates the loop linking material properties to site procedures. The choice of mix proportions is therefore an interactive process where site trials constitute an essential element. The experience from similar projects and from field experience with the proposed constituents provides valuable input and the recommendations contained in the present report should be seen in this light.

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[1] Draft report on "Concrete mix design," by Aker Maritime, Tom Moksnes, for the Office of Naval Research, Feb 1999.

It should also be emphasized that the chosen mix proportions must be sufficiently robust to stand up to changing site conditions. High performance concrete is generally sensitive to fairly minor changes in material properties and procedures. A 'hands on' approach to concrete production with immediate response and adjustments to the mix as the need arises imposes requirements on the efficiency of the quality control system and the competence of the operators.

In the past there was a tendency to focus on the strength properties at the expense of equally important properties such as durability, workability, and weight.

DURABILITY

Concrete in seawater has adequate and satisfactory durability provided attention is paid to a few simple rules of mix design and construction. The essential requirements are the use of high quality concrete with low permeability and high frost resistance, and adequate and uniform concrete cover to the reinforcement.

Corrosion of the reinforcement is the most fundamental concern in concrete sea structures (rather than concrete deterioration), and ensuring that the steel is surrounded by adequate thickness of low permeability concrete will prevent corrosion. In simple terms, corrosion will not occur if neither oxygen nor seawater carrying chlorides can penetrate to the steel and cause a lowering of the pH and the creation of an electrochemical circuit.

> **The quality and minimum thickness of the concrete cover is the major durability factor.**

Cracks caused by shrinkage, creep, thermal gradients, and loads may also render the steel vulnerable to seawater ingress and subsequent corrosion and must be dealt with during design and construction. These factors are affected by the concrete composition as well as by the site procedures related to placing and curing. Adequate water spray or membrane curing should always be applied to the fresh concrete.

Other durability concerns for concrete in sea structures may include alkali-aggregate reactivity (AAR), sea water reactivity (sulphate and chloride reactions with the cement compounds), and frost resistance in the case of concrete exposed to freezing and thawing. The total chloride content in the fresh concrete needs to be restricted and the mixing water should always be fresh and potable.

For marine structures and particularly for unknown aggregate sources, petrographic examination by thin section microscopy should always be performed

Concrete deterioration due to sea water is the result of separate reactions between sulphates and chlorides present in the sea water and the cement compounds tricalcium aluminate C_3A and portlandite $Ca(OH)_2$. A well-compacted high performance concrete with a low w/c ratio will provide adequate durability in the marine environment. Cement with a low C_3A content is recommended for concrete exposed to aggressive types of sulphate attack. Some caution should be exercised in less aggressive environments due to the effect this may have on other and equally important concrete properties. A moderate C_3A content of 5 – 7 % is commonly specified for concrete sea structures and has worked well.

Freeze-thaw resistance is dealt with by air entrainment, using air entraining admixtures to achieve the desired size and spacing of the air voids. It should be kept in mind that air entrainment reduces the compressive strength and that stable air voids are not always easily achieved in highly workable (superplasticized) concrete.

CONSTRUCTABILITY

Pumping is an expedient and cost efficient method of placing concrete in large and tall structures and pumpability of the concrete becomes a key factor in the mix design. Also, tall structures are often built by slipform construction that imposes special requirements on the fresh concrete. Finally, high strength concrete is commonly used for structures that are densely reinforced and prestressed and therefore demand highly workable mixes. This adds up to the fact that the properties of the fresh concrete are essential considerations in the mix design process. The main quality parameters for fresh concrete, commonly labeled constructability, are:

Workability (flow characteristics)
Stability (lack of segregation and water separation)
Open time (pot life)

Essentially, what is required is a high slump (superplasticized concrete), a cohesive mix with little or no bleeding and segregation during pumping, compaction, and setting, and a mix with the ability to retain its fresh characteristics until the mix is placed and compacted. The setting time, i.e., the time after which the concrete can not be vibrated or remoulded and starts to harden, is critical for the rate of slipforming and is significantly affected by the choice and dosage of admixtures. A summary of the main quality challenges and the chosen remedies is shown in Figure 2-1.

HIGH STRESSES 30m design waves. High hydrostatic pressures. Dynamic loads (earthquakes). Severe loading in some construction phases, e.g., extreme water pressure on the cell walls during submergence for mating.	**REQUIREMENTS:** High performance constituent materials of uniform and predictable quality. Characteristic 28 day cube strength up to 80 MPa. Acceptable fatigue properties
DEMANDING CONSTRUCTION Dense reinforcement (300 kg/m^3 +). Large volumes, tight tolerances. Pumping to heights over 200m. Large scale slipform construction.	High slump (250 mm) No bleeding or segregation. Acceptable and verified pumpability. Adjustable and predictable setting time.
HOSTILE ENVIRONMENT Sea water Steel corrosion. Concrete deterioration. Freeze thaw damage.	Low w/(c+s) ratio (0.35 – 0.40). Cement (binder) content min.400 kg/m^3 in splash zone. Permeability coeff. 10^{-12} m/sec. Water intrusion (ISO/DIS 7031) 25mm (target 15mm) Curing temp. max. 65-70^{o}C. Concrete cover min 50mm to ord. Reinforcement, min 70mm to prestressing steel. Air entrainment in freeze/thaw conditions.

Figure 2-1. The main concrete quality requirements adopted in the North Sea [2]

CEMENT

Different cements will have different water demands for a given cement paste consistency and will therefore affect the rheological properties of the concrete mix. The C_3A content of the cement affects several properties and the suggested value of 5.5 is a compromise between its effect on sulphate resistance, chloride initiated rebar corrosion, heat of hydration, and early loss of slump.

A major step in the mix design process is to identify a suitable cement or cement/pozzolan combination for the prevailing high performance concrete requirements.

AGGREGATES

Aggregates constitute 70% of the volume of the concrete mix. For low or medium strength concrete a wide range of natural or crushed aggregates will have adequate properties provided they comply with the mandatory requirements. This is not the case for modern high strength/high performance concrete where the strength of the aggregate particles will affect the ultimate strength and deformation characteristics of the chosen mix. Similarly, the particle size, grading, and mineral composition will significantly affect the water content required for a given workability, and hence the w/c ratio and durability.

The properties of the aggregates depend on their geological origin, their geological history in terms of transportation and sedimentation, and their quarrying, processing, and handling methods.

This development of tailoring the sand grading, including the content of silt and fines, has had a profound effect on the properties of the fresh concrete, the w/c ratio, and the strength properties. These properties are also affected by the particle size, shape, and texture of the coarse aggregate which can also be modified by selected processing methods. The mechanical properties of the aggregates play an important role in controlling the strength, E-modulus, ductility, and fracture mechanics properties of high strength concrete. Variations in the E-modulus of the coarse aggregate may significantly affect the E-modulus of the concrete (by as much as 25 to 50%). For equal uniaxial compressive strength levels, different rock types may cause 15 to 40% differences in tensile and flexural strength.

The durability aspect concerning aggregates is linked to their chemical stability and to their secondary role in determining the concrete density through their effect on w/c ratio and workability. A main concern regarding chemical stability is the ability to identify potentially alkali reactive aggregates and thin section microscopy should be conducted on any unknown source of aggregates.

LIGHTWEIGHT AGGREGATES

[2] A.K.Haug, M.Sandvik. Mix design and strength data for concrete platforms in the North Sea. 2. International Conference on Performance of Concrete in Marine Environment, St.Andrews, Canada, August 1988.

Different types of LWA concrete have been developed and been found to enhance properties such as durability and energy absorption. LWA concrete comes in a wide range of densities and strengths and there are no clear lines of division separating LWA concrete from normal density (ND) concrete.

Some lightweight aggregates with lower water absorption may be suitable for pumping provided the aggregates are thoroughly soaked in water prior to mixing. One such material is Stalite, a proprietary material of the rotary kiln expanded shale type produced in the U.S. Stalite was used on the Hibernia GBS project in Newfoundland for the production of large quantities of 80 MPa MND concrete.[3]

The high porosity and water absorption of the LWA aggregates is a major factor in mix design and production. The water absorbed during mixing must be uniform and predictable to achieve consistent properties.

LWA concrete has lower thermal conductivity and lower heat storage capacity than ND concrete and the temperature rise due to heat of hydration will be higher. Cooling measures may need to be implemented in large sections.

All the design properties of high strength LWA concrete have been studied and many differ to some extent from those of ND concrete. A summary of the general trend that has been observed is given below in Figure 2-2:

Advantages of LWA Concrete	**Disadvantages of LWA Concrete**
Reduced weight and improved buoyancy. Better crack behaviour from shrinkage, creep and thermal expansion. Reduced cracking from deformation loads. Better energy absorption from impact loads. Lower permeability. Improved durability and corrosion resistance. Improved freeze-thaw resistance. Equal or better fatigue behaviour.	Reduced resistance to locally concentrated loads and need for confining reinforcement. Lower E-modulus, brittle failure mode. Higher cement content for given strength. Higher heat of hydration. Liable to spalling of cover under HC fire. Need to control water content and absorption for consistent workability. More demanding to batch, place and cure. Higher cost per m^3.

Figure 2-2. Observed relative merits of high strength LWA compared to ND concrete.

It should be emphasised that the test results and observations reported above need to be verified for the chosen concrete mix design and lightweight aggregate. LWA concrete is well suited for marine applications provided all the different properties, such as the increased brittleness, are accounted for in the detailed design of the structure.

[3] C.G.Hoff, R.Walum, J.K.Weng, R.A.Nunez: The Use of Structural Lightweight Aggregates in Offshore Concrete Platforms, Proceedings International Symposium on Structural Lightweight Aggregate Concrete, Sandefjord Norway June 1995, Norwegian Concrete Association, Oslo.

ADMIXTURES

Chemical admixtures, and particularly the water reducing types, are essential to the production of high strength/high workability/low w/c ratio concrete. Major improvements in their performance have been achieved in recent years.

Today, a range of very efficient proprietary high resolution water reducing admixtures (HRWRA) are available and are used in dosages of 1 - 2% of the cement weight. Site trial mixes need to be performed to decide the best product(s) and dosage for the specific purpose as the total mix composition and the batching process and procedure may affect the results.

If air entrainment is deemed necessary to achieve frost resistance in freeze/thaw conditions, air-entraining admixtures can be used to obtain the desired volume of very small micropores. Acceptable frost resistance is commonly evaluated by measurement of the air void system or by freeze/thaw test procedures. The need for freeze/thaw protection of high strength concrete in the North Sea is an ongoing discussion and research suggests that non air entrained low w/c ratio concrete has a high resistance to freeze/thaw deterioration.

Air entrainment will lead to a loss of compressive strength, and this must be taken into account when air entrainment is contemplated and decided. Air entraining admixtures are commonly used in small dosages of 0.1% of the cement weight and work better with some HRWR admixtures than with others.

Set retarding admixtures are used when the rate of construction is such that the set retarding effect of the HRWR admixtures is not sufficient. Chloride free set accelerators have also been developed for a quicker set but have so far only had limited application for the large offshore projects.

Common to all admixtures is that their performance is dependent upon a number of mix design and site specific factors and that they are marketed under proprietary brand names. Site trials are therefore essential prior to selecting the most efficient brands and dosages.

SILICA FUME

Modern high strength concrete benefits by a small dosage of silica fume and is considered essential for high strength MND and LWA concrete.

Silica fume (microsilica) is a by-product of the ferro-silicon industry. It is an extremely fine powder with grain size less than 0.1 m and specific surface of about 20,000 m^2/kg. It is a reactive pozzolan, consisting of 90% amorphous SiO_2 and acts as a very effective filler. Its fineness and its reactivity account for the beneficial effects of silica fume in high performance concrete.

Silica fume in moderate dosages will increase the compressive strength, increase the resistance to corrosion of the embedded steel, and improve the resistance to segregation of a high slump concrete. Dosages of 5 – 8% are considered very beneficial for all high performance LWA concrete. Silica fume can be blended into the cement at the mill or distributed and added as a slurry during batching. There are side effects, particularly at high dosages, related to stickiness and plastic cracking if proper curing measures are not implemented.

FIBER REINFORCEMENT

Fibers may be used to increase the tensile strength and the toughness of concrete, an example of the improved ductility that can be achieved by the addition of 1% fiber. Fibers will also increase the impact strength and reduce shrinkage. The fibers may be steel, carbon, glass, polymers, and other synthetic materials. The length, shape, and mechanical properties of the fibers affect the way they impact on the properties of the concrete. Fiber contents of 1 – 5% by volume have been used for special applications such as sprayed concrete, precast units, special hard wearing pavements and caps for driven piles.

The inclusion of fibers significantly affects the properties of fresh and hardened concrete and introduces special requirements on the batching plant and procedures. Fibers have not been used for the mass concrete on any of the large offshore projects described in this report. Although they may appear to offer advantages as 'crack reinforcement', their impact on constructability and on construction procedures is negative. The addition of even small quantities of fiber reinforcement should be considered very carefully and thoroughly tested on site before they are adopted for improved ductility and durability of large marine concrete structures.

COATINGS

Polymer impregnation of concrete may improve the resistance to freezing and thawing, the abrasion resistance, and the general durability in an aggressive environment. The application is difficult and demanding and would not be practical to perform on very large structures. Polymer coatings have not been applied to the offshore projects described in this report. Polymer modified mortar coatings were, however, applied in some critical areas on the Northumberland Strait Bridge Project.[4]

Epoxy coatings have been applied to some of the North Sea structures to provide additional protection in the splash zone. The epoxy was applied by trowel to the freshly slipformed surfaces of the shafts and subsequent inspections have revealed that the coatings have been successful.[5]

THE SIGNIFICANCE OF THE W/C RATIO

The general relationship between the w/c ratio and compressive strength is good for the 0.35 to 0.40 range of w/c ratio commonly specified for offshore concrete structures. Above a w/c ratio of 0.40 the volume of capillary water and air pores increases rapidly and contributes to a more porous cement paste and reduced durability.

Permeability tests show that the coefficient of permeability of cement paste increases exponentially as the w/c ratio increases above 0.50.[6] The low permeability associated with low w/c ratios gives added durability which in some cases is a more significant feature of high performance concrete than the strength itself.

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[4] E.W.Tromposch, L.Dunaszegi, O.E.Gjørv, W.S.Langley: Northumberland Strait bridge project-strategy for corrosion protection. Proceedings 2. International Conference on Concrete under Severe Conditions, CONSEC 98 Tromsø, E&FN Spon, London.

[5] R.Aarstein, O.E.Rindarøy, O.Liodden: Effect of Coatings on Chloride Penetration into Offshore Concrete Structures. Proceedings, CONSEC 98, Tromsø. E&FN Spon, London.

[6] A.M.Neville, J.J.Brooks: Concrete Technology, Chapter 14, Longman, London 1987.

The low w/c ratio benefits both strength and durability and the range of 0.35 to 0.40 is commonly adopted for high performance concrete. A w/c ratio below 0.35 requires high binder contents and very high dosages of HRWR admixtures to achieve a satisfactory workability and may have undesirable side effects with respect to early cracking. Such concrete has also been found to be vulnerable to inadequate site procedures and to require more skilled and experienced operators.

SUMMARY

No mix design can be completed without site experience and full scale site trials. The chosen mix proportions for high performance concrete need to be continuously monitored and adjusted as the work progresses to account for variations in the local conditions. From a virtually total focus on *high strength* working close to the level of feasibility, industry is shifting their focus to *high performance.*

It should be remembered that high strength concrete is less tolerant to variations in site conditions and practices than ordinary concrete and the quality obtained depends to a larger extent on the knowledge and skill of the operators.

The designer has the freedom to choose within a broad range of options to achieve the desired properties of the fresh and hardened concrete while still operating within the realm of high strength/high performance concrete. The fundamental requirements for durability are high quality constituent materials, a low water/binder ratio, good constructability facilitating placing and compaction, proper curing, and adequate cover to the steel.

The term “high quality constituent materials” requires special attention and implies thorough durability testing according to the relevant Codes as well as mechanical testing where compliance with the Code is only the first step in the process. Full scale site tests on concrete mixes and mock-up tests on site procedures will be needed for a finalization of the concrete mix design. Once the mix constituents and proportions have been selected, a quality assurance system needs to be established and implemented that ensures continuous monitoring and adjustments to the mix to suit the prevailing and changing conditions on site. An important aspect of this system is the interface with the site procedures for formwork and rebars to ensure that the concrete can be properly placed and compacted and that the specified concrete cover can be maintained.

CHAPTER 3

USE OF NEW GENERATION EPOXY-COATED REBAR IN THE ADMIRAL CLAREY BRIDGE

USE OF NEW GENERATION EPOXY-COATED REBAR
IN THE ADMIRAL CLAREY BRIDGE

INTRODUCTION

The design and construction of the Admiral Clarey Bridge exemplifies the use of durable reinforced concrete in a marine environment. Planners, designers, and builders must pay great attention to the many critical factors that ultimately contribute to the durability of the reinforced concrete. This paper provides a brief summary of some of the important concrete material issues related to performance with particular emphasis on supplemental corrosion protection using new standards for prefabricated epoxy-coated steel rebar.

Construction of the Admiral Clarey Bridge, Pearl Harbor, Hawaii

THE ADMIRAL CLAREY BRIDGE

The Admiral Clarey Bridge connecting Ford Island to the Pearl Harbor Hawaii Naval Complex was dedicated April 15, 1998. The 4,700-foot long bridge is one of six reinforced concrete floating bridges in the world. The 650-foot moveable span is the longest in the world.

The request for proposals (RFP) for the design/build contract was developed with the assistance of a number of people and organizations. For the bridge concept, the RFP relied heavily on studies by the Pacific Division Naval Facilities Engineering Command's (PACNAVFACENGCOM) Planning Department. These included various Ford Island Access studies accomplished in 1987/88 and the Final Environmental Impact Study in 1990.

The RFP provided specific design criteria, which were developed mostly by PACNAVFACENGCOM's Design Division engineers with assistance from the following:

Naval Facilities Engineering Command, John Headland, Coastal Engineering.

Washington State Department of Transportation, Myint Lwin, floating pontoon section and concrete.

Federal Highway Administration, Raymond McCormick, highway/bridge.

Naval Facilities Engineering Service Center (NFESC), Douglas Burke, prefabricated fusion-bonded pipeline-type epoxy-coated reinforcement.

Because rebar corrosion occurs much faster in a tropical environment, such as Hawaii, it was particularly important that emphasis be placed on the design of the concrete materials to provide long term durability. To maximize concrete durability, these design decisions were made:

The use of 5 percent silica fume was recommended by Mr. Lwin based on his experience and success in using silica fume on the most recently constructed Washington State floating bridges.

The use of a maximum allowable water-to-cement ratio (w/c) of 0.38 was based on waterfront engineering practices using locally available Hawaiian concrete materials.

The use of a zero tension under service load criterion was based on the State of Hawaii Department of Transportation requirement for all bridges in Hawaii.

The use of prefabricated fusion-bonded epoxy-coated rebar was a difficult decision to specify since the Navy's new standard was still under development by NFESC and the increased cost was uncertain. Ultimately, 4,600,000 pounds of epoxy-coated mild reinforcing steel was used to construct the bridge.

COST/BENEFIT

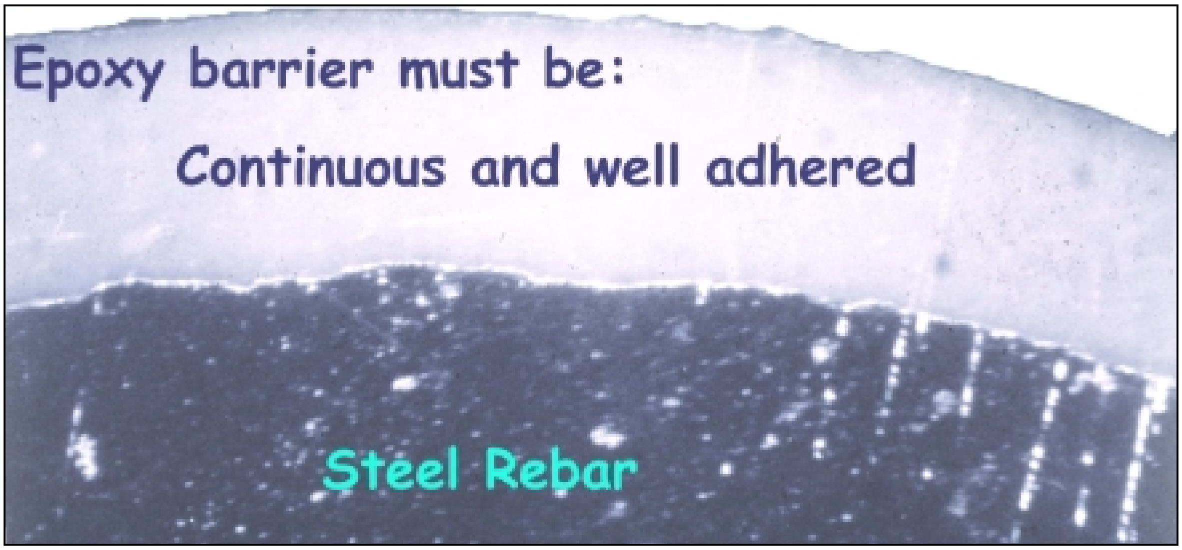
For the Admiral Clarey Bridge a cost comparison of using plain steel rebar versus prefabricated epoxy-coated rebar was done. The predicted costs were $1.20/pound for plain rebar compared to $1.60/pound for coated rebar, installed. Therefore the additional cost amounted to ($0.40) x (4,600,000) = $1,840,000. Since the cost of the total project was $86 million, the premium to use this technology was 2.1 percent. Use of high quality concrete materials and workmanship with proper concrete cover should provide a 50-year service life. Field performance evaluations and accelerated laboratory tests of coated rebar indicate that the technology will provide a substantial increase in life performance. Rebar life extension on the order of 20 to 40 years is a rational expectation.

EPOXY-COATED REBAR DEVELOPMENT

The decision to use epoxy-coated rebar in new Navy construction was based on extensive evaluations that began in 1984, when the Office of Navy Research tasked the Naval Civil Engineering Laboratory to conduct long-term field evaluations. Test specimens were suspended in a marine intertidal zone for 76 months at Key West, Florida to rank the relative performance of popular corrosion control methods. Damage-free epoxy-coated rebar performed best. Results from this study were presented by the Concrete Reinforcing Steel Institute in their Research Series 2 report of July 1994.

Despite the good performance in the Navy's long-term field tests, the Florida Department of Transportation and other agencies had found moderate to severe corrosion much earlier than expected on some marine structures using epoxy-coated rebar. By 1994, much controversy surrounded the use and performance of epoxy-coated steel reinforcing bars produced and placed in accordance with current specifications. Consequently, the Navy Criteria Office funded the NFESC to identify the failure mechanisms in current practices and to develop a new standard in cooperation with industry experts. This effort resulted in an Interim User's Guide for Prefabricated Epoxy-Coated Rebar for Oceans and Other Severe Environments (PROSE). The document included two new Navy Facilities Guide Specifications (NFGS), 03201 and 03202, and recommendations for a quality control program. The Navy Criteria Office identified candidate construction projects to incorporate the new generation of epoxy-coated rebar. Two Navy submarine piers were constructed, one in Pearl Harbor, Hawaii, and the other in New London, Connecticut. NFESC monitored the construction of each project and evaluated the cost and constructablity. Both projects proved highly successful and the differential costs were about 2 percent higher for each with respect to the overall construction cost. The toughness of the new epoxy powder formulation developed by 3M proved exceptionally good, requiring very few repairs after shipping, storage, and placement. The bridge also included small sections of epoxy-coated rebar coated with epoxy powder formulated by Akzo Nobel and Herbert’s-O'Brien, which appeared to be equally durable.

The American Society of Testing and Materials (ASTM) used the Navy's draft specifications as a basis for the development of ASTM A 934/A 934M published in July 1995, “Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars.” Mr. D. Burke of NFESC is the current chairman of the ASTM Subcommittee A01.05 task group for development and revision of coated reinforcement standards. In February 1998, the NAVFAC Criteria Office published, for the first time, a definitive guide for Marine Concrete, NFGS 03311. Included is a requirement to use prefabricated epoxy-coated reinforcing steel according to the new ASTM Standard.



Magnified View of Epoxy-Coated Rebar

IMPORTANT FEATURES FOR ENHANCED PERFORMANCE

There are many important features of the new technology for prefabricated epoxy-coated rebar contained in the ASTM Standard that contribute to improved performance. Some of these are:

All of the rebar is prefabricated to final size and shape prior to coating. This avoids stress cracks in the coating and loss of coating adhesion in the bend areas during post fabrication, which has been a typical site for corrosion.

Since the coating no longer needs to be flexible, new epoxy powder formulations can be used. These formulations are more durable and resistant to the intrusion of corrosive elements.

Extensive quality control tests must be performed on every batch of coated rebar, including cathodic disbondment tests for coating adhesion. This requirement greatly reduces problems with underfilm corrosion.

All visible defects in the coating must be repaired prior to concrete placement. This minimizes the number of locations in the barrier coating that might otherwise become corrosion sites.

In addition to the recommendations contained in the ASTM standard, NFESC strongly recommends that:

Coated rebar is not mixed with plain rebar in the structure. This avoids the possibility of creating a large corrosion cell if there is electrical continuity between the coated and uncoated steel.

Coated rebar should not be used in structures that are subject to large impact loads and in areas where the steel is severely congested (e.g., 50 percent or more of the cross section is steel). Because of the lack of adhesion of the cement paste to the epoxy coating, the concrete that covers the reinforcement may disbond when subject to impact loads, which was reported when a reinforced concrete component was accidentally dropped.

Designers should not specify the use of coated rebar that exceeds number 11 (2-3/4" diameter) until definitive data is available that addresses the effect on bond and anchorage.

CORROSION ACTIVITY

The purpose of providing supplemental corrosion protection, such as an epoxy coating, is to reduce the rate of rebar corrosion, thus increasing the time before corrosion related repairs are necessary. This is accomplished in two important ways:

If the quality of the concrete is compromised in any manner that results in cracks, increased concrete permeability, or reduced concrete cover, then chloride, oxygen, and water will find their way to the rebar sooner than expected. An excellent barrier coating on the steel will extend the time before corrosion will take place.

Eventually corrosive elements will reach the rebar regardless of the concrete materials used and the quality of the workmanship. When the chloride contamination reaches the threshold level necessary for the initiation of steel corrosion, the presence of a highly impermeable well-adhered barrier coating with a minimum number of defects will retard the potential for corrosion activity in the steel reinforcement.

CONCLUSION

Concrete durability in a marine environment requires strict attention to many important aspects of planning, materials, design, and workmanship. Life performance of marine structures can be enhanced by the use of prefabricated epoxy-coated steel reinforcing bars with good coating adhesion and no visible damage to the coating. Construction of the Admiral Clarey Bridge exemplifies the use of these design and construction principles.

OTHER PROJECTS USING EPOXY-COATED REBAR

Several projects within the Navy and in the private sector have used the new standards for prefabricated epoxy-coated rebar, such as, the Muni-Metro Turn Back in San Francisco, California and the Long Beach Aquarium in Long Beach, California. In June 1997, the technology was reviewed and adopted by the California Department of Transportation (CALTRANS) for reinforced concrete structures in contact with sea and brackish water.

Construction of Muni-Metro Turn Back, San Francisco, California

ADDITIONAL INFORMATION

Additional information about the design and construction of the Admiral Clarey Bridge is contained in an excellent and comprehensive article by Michael Abrahams and Gary Wilson featured in the PCI Journal July/August 1998 issue. For more information about the use of prefabricated epoxy-coated reinforcement, please contact Douglas Burke at 805-982-1055 or burkedf@nfesc.navy.mil.

CHAPTER 4

CALCIUM NITRITE ADMIXTURE

CLACIUM NITRITE ADMIXTURE

BACKGROUND

Due to the severity of a marine environment and the likelihood for corrosion of the steel reinforcement, designers must specify high quality concrete and an adequate concrete cover over the rebar. In addition, the use of a corrosion protection system can provide for additional corrosion protection. Epoxy-coated steel rebar, galvanized steel rebar, and calcium nitrite admixture all provide beneficial effects. ("Performance of Epoxy-Coated Rebar, Galvanized Rebar, and Plain Rebar with Calcium Nitrite in a Marine Environment," D. Burke, July 1994.)

PASSIVATION OF STEEL IN CONCRETE

Due to the alkaline environment of concrete, a protective passive layer of iron oxide forms on the surface of the steel rebar. This passive layer is composed of ferrous (Fe^{2+}) and ferric (Fe^{3+}) oxides (refer to Figure 1). Ferrous oxides are susceptible to chloride attack, whereas, ferric oxide resists chloride attack. It is important to note that the ingress of carbon dioxide, water, and oxygen can also contribute to the breakdown of the protective passivation layer. This is called carbonation corrosion.

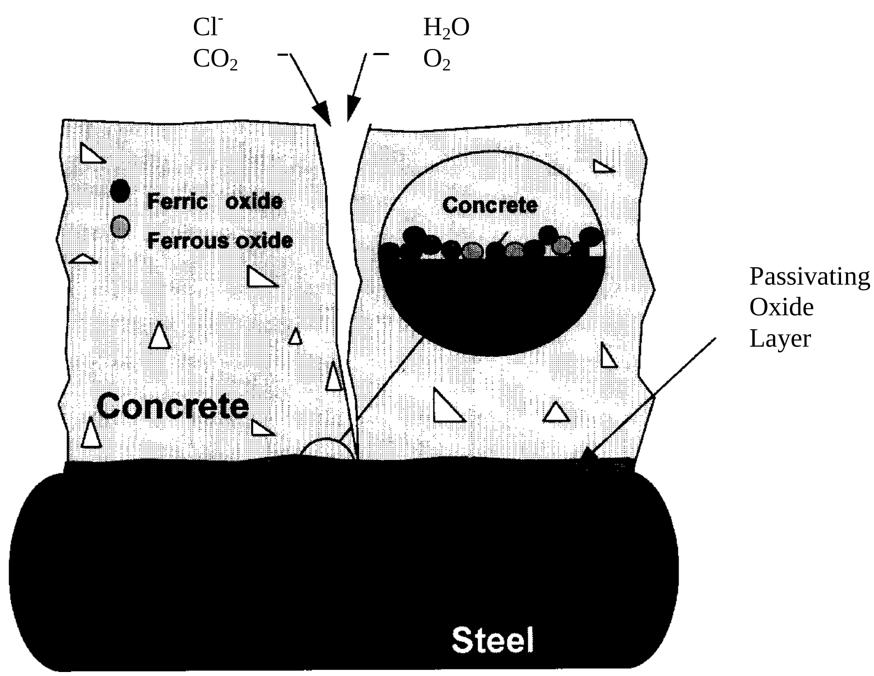


Figure 1. Passivation of steel in concrete.

CHLORIDES AND CORROSION

Corrosion of steel reinforcement in concrete occurs in locations where chloride intrusion has weakened and/or destroyed the passive layer. This commonly occurs at the ferrous oxide sites.

The chloride level necessary to initiate corrosion of steel in concrete is approximately 1.5 lb/yd^3. The time it takes to reach this level of contamination at the depth of the rebar is dependent on the exposure, quality of concrete, depth of concrete cover over the reinforcement, and the number and depth of cracks that develop in the concrete. Due to the high availability of chloride ions in a marine environment, the chloride corrosion threshold can be reached within a few years.

CALCIUM NITRITE ADMIXTURE

To protect the rebar and passive layer from attack, calcium nitrite can be introduced into the concrete mix during batching as an admixture. It is introduced into the concrete at either the precast or cast-in-place concrete operation. Calcium nitrite reduces the steel rebar's susceptibility to corrosion by increasing the oxide surface concentration, hence strengthening the passive layer. The duration of protection offered by calcium nitrite is dependent on the dosage used and corrosion rate.

The amount of calcium nitrite the specifier selects is a function of concrete quality and the environment to which the structure is to be exposed. Therefore, these factors need to be considered when selecting a dosage of calcium nitrite. General guidelines as to the dosage amount for certain environments are provided by the manufacturer. A marine environment typically has a recommended calcium nitrite (30% solution) dosage of 4 to 6 gal/yd^3. The most common dosage is 4.5 gal/yd^3 of concrete. American Concrete Institute has published two applicable references: ACI 212.R-91 Chemical Admixtures for Concrete and ACI 222-89 Corrosion of Metals in Concrete.

Calcium nitrite has not detrimental effects to hardened concrete properties. Both neutral set and accelerated set versions are available to accommodate project requirements.

CAST-IN-PLACE CONCRETE

Cast-in-place concrete is frequently used in the construction of seawalls, piers, and berthing docks. Cast-in-place concrete quality is dependent on many factors. Some of these factors are water-to-cement ratio (w/c), total cement content, aggregate type and gradation, curing conditions, and job site weather conditions. A corrosion protection system is required for direct marine exposure. When the use of more than one protection system is specified for the same structural component, this is referred to as a redundant system. The use of a redundant corrosion protection system properly addresses the durability required for these marine-exposed structures. Calcium nitrite and epoxy-coated rebar is an example. The calcium nitrite helps protect the steel where defects occur in the coating. One may wish to use a redundant system to increase confidence in long term durability.

PRECAST CONCRETE

Precast concrete is commonly used in marine piles and substructure deck members. Due to the controlled manufacturing conditions, precast members are typically of high quality and uncracked. In use, these concrete members are exposed to direct sea water and therefore will benefit from supplemental corrosion protection. The use of epoxy-coated rebar and a calcium nitrite admixture should be considered.

CONCLUSIONS

Research has shown that the addition of calcium nitrite admixture to concrete can slow the onset of corrosion for both cracked and uncracked test specimens. Calcium nitrite admixture has been used commercially since 1978. The dosage of the product needs to be determined for each project based on the predicted chloride diffusion rate and desired design life. To be successful with calcium nitrite, be sure to follow the manufacturer's recommendations and instructions for application. All marine concrete should use quality concrete with a low water-to-cement ratio concrete (<0.40), proper concrete covers, and proper curing to maximize performance. These recommendations are consistent with ACI concrete practice guidelines, which should be followed.

POINT OF CONTACT

If you would like further information on this subject, contact Mr. Douglas Burke, Naval Facilities Engineering Service Center, Code ESC63, at (805) 982-1055 or DSN 551-1055, or e-mail at burkedf@nfesc.navy.mil.

CHAPTER 5

NFGS-03311 – MARINE CONCRETE

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DEPARTMENT OF THE NAVY   NFGS-03311
NAVAL FACILITIES   15 February 1998
ENGINEERING COMMAND
GUIDE SPECIFICATION
****

NFGS-03311

MARINE CONCRETE

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Preparing Activity: NFESC EC DET (00CE4)

	Typed Name & Reg.	Signature	Date
Prepared by:	M.S. Chawla, P.E.	____________	______
Approved by:	P.N. Bolton, P.E.	____________	______
Approved for NAVFAC:		____________ Carl E. Kersten, R.A.	______

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Note: This document is available through John Lynch, Criteria Office, Code 15, EFD, LANTDIV. Phone: 757-322-4207, DSN 262-4207; e-mail Lynchjt@efdlant.navfac.navy.mil.

ABS CLASSIFICATION GUIDE
CONCRETE MATERIALS FOR DoD MOBILE OFFSHORE BASE (MOB)

Prepared by:
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This draft revised on: 17 January 1999

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10.3.5 Concrete Materials for Aircraft Traffic Surfaces

10.3.5 Quality Control:

10.3 Reinforced Concrete for MOB

The reinforced concrete used in MOB construction shall be of sufficient durability and strength to meet the intended service conditions and performance life as expressed in the MOB Mission Requirment Statement. A satisfactory methodology for design, material selection, placement, and quality control shall be developed. Consideration shall be given to the use of prestressed lightweight concrete and modified density concrete. Development of the criteria and specification shall reflect international performance records, codes, standards and research data.

10.3.1 General Requirements

10.3.1.1 Scope

The scope of this section is to set forth requirements for prestressed lightweight concrete and modified density concrete to construct the MOB. These requirements are also applicable for a "hybrid" MOB, constructed with structural steel and prestressed concrete. Emphasis is placed on the use of structural modified density concrete and a methodology for material selection that will result in a durable concrete structure. The term "concrete MOB" implies a concrete hull using columns and pontoons that support a steel deck. On top of the steel deck, the aircraft-wearing surface may also be constructed with concrete. Design and construction of the MOB will present unique challenges; issues related to design and construction are addressed in other documents. [1,2]

10.3.1.2 Exposure Conditions

The MOB will operate in a severely aggressive environment and will be continuously exposed to ocean currents, wave action, seawater spray, tidal action and submerged conditions. Concrete exposure for MOB may be determined from the exposure classes defined in appropriate standards.[3,4,5,6] Materials criteria and specifications shall be developed to produce durable concrete to resist the failure mechanisms summarized in section 10.3.1.4 Design Methodology.

10.3.1.3 Durability Requirements

The use of prestressed concrete and reinforced concrete has been found appropriate for construction of ships and offshore structures in severe marine environments. [7,8,9] The United

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[1] U.S. Navy Mobile Offshore Base ARCOMS Concept Study

[2] Structural Analysis and Design, 7 April 1998, Document No. 9019-ANC-JD-RN-0005

[3] Building Code Requirements for Structural Concrete, ACI 318, Chapter 4. American Concrete Institute POB 9094 Farmington Hills, MI 48333.

[4] British Standards Institution (1997a). pr EN206. Concrete - performance, production and uniformity. Draft for Public Comment, BSI Document 97/104685, Committee Reference B/517.

[5] Norwegian Standard NS 3473 E

[6] Hobbs, D.W., Minimum requirements for durable concrete, British Cement Association 1998

[7] ACI, State-of-the-Art Report on Barge-Like Concrete Structures Reported by ACI Committee 357, ACI 357.2R-88

States Navy commissioned the construction of at least 15 concrete ships during World War I including the USS Selma, USS Atlantis and USS Polias. During World War II more than 100 concrete ships were constructed. Several of these ships have been the subject of material durability investigations that have concluded that concrete can provide long term durability in a marine environment. [10,11] However, "many structures built in accordance with codes and guidelines of recommended practice have shown deterioration much before their intended service life".[12] Therefore, it is mandatory to incorporate a high level of quality in design, material selection and construction. To accomplish the necessary durability it is recommended that the designer take a holistic approach to durability.[13] In addition, the development of the criteria and specifications should incorporate recommendations contained in the following documents.

ACI 357 Concrete for Offshore Structures
ACI 357.2R Concrete for Barge-like structures
Canadian Standards S-374 Structures for Offshore and Frontier Areas, Concrete
ACI committee reports on structural lightweight aggregate concrete[14,15]
ACI 318 Building Code[16]
U.S. Navy specifications for marine concrete[17]
European Standard pr EN 2061 (draft)
Norwegian Standard NS 3473 E.
CEB Bulletin 238 - New Approach to Durability Design
FIP Manual of Lightweight Aggregate Concrete

[8] Severin, L. et. al. "Troll A Gas Production Platform – Implications of 50-70 Years Life Span, OTC 8412, Offshore Technology Conference, 1997

[9] Anderson, A. R., " A 65,000-Ton Prestressed Concrete Floating Facility For Offshore Storage Of LPG"

[10] Heun, Raymond C., "Concrete Ships -- Long Forgotten," Concrete International, April 1995, pp. 54-56.

[11] Bremmer, Theodore W.; Holm, Thomas A.; and Morgan, Dudley R., "Concrete Ships -- Lessons Learned," Performance of Concrete in Marine Environment, AP-163, American Concrete Institute, 1996, pp. 151-169

[12] Mehta, P. K. "Point of View, Durability – Critical Issues for the Future", Concrete International, July 1997, pp. 27-33.

[13] Mehta, P. K. and Gerwick, B.C., "Concrete in the Service of Modern World," Proceedings of the International Conference on Concrete in the Service of Mankind, University of Dundee, Scotland, June 1996.

[14] ACI Committee Report 304 "Batching, Mixing, and Job Control of Lightweight Concrete", 1991.

[15] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Draft October 1996.

[16] ACI Committee Report 201.2R "Guide to Durable Concrete," Manual of Concrete Practice, American Concrete Institute, Farmington Hills, MI, 1997 pp. 33-37.

[17] Naval Facilities Engineering Service Center, Naval Facilities Guide Specification NFGS-03311, February 1998

10.3.1.4 Failure Mechanisms

A long service life with no major repairs in a marine environment demands the use of high quality reinforced concrete. The design methodology used shall account for the failure mechanisms that may manifest as deterioration of the concrete. Typically, the most likely deterioration mechanism for marine exposure is corrosion of the steel reinforcement. The concrete must be resistant to deterioration from the following mechanisms over the performance life of the structure.

Concrete shall be of high tensile strain capacity to be able to resist the formation of cracks due to the volume changes, and shall be reinforced and/or prestressed to provide adequate toughness to resist the crack propagation due to the various mechanical, physical and environmental loadings. Concrete shall be resistant to chemical disintegration caused by alkali-aggregate reactions, sulfate attack, and delayed ettringite formation. Concrete shall have adequate resistance to freezing and thawing attack. Resistant to wear and to the formation of cracks and crack propagation from impact loads, chains, cables, minor collisions, wave action, vibration, fatigue, abrasion, freeze and thawing, and other physical and environmental loading. Resistant to the ingress of environmental agents such as air, water, chlorides, sulfides and carbonates.

10.3.1.5 Basis of Design

"One important limitation of conventional concrete, even of good quality, is the presence of microcracks, capillaries and micro-capillaries into which water is able to penetrate; sucked in by surface tension forces or driven by an external hydrostatic pressure."[18] Intrusion of water is a significant factor affecting the rate at which many concrete failure mechanisms progress. The use of high performance lightweight concrete or modified density concrete can provide superior impermeably to structure subject to hydrostatic pressure when compared to normal weight concrete..[19] A durable concrete MOB must be highly resistant to the ingress of moisture. Chloride ions also migrate through permeable concrete by diffusivity, even without actual flow of water. Therefore, the design and materials selection must strive to minimize the ingress and migration of water, oxygen, chlorides, sulfides, and carbon dioxide into the concrete. In time, these substances are directly responsible for the deterioration of the concrete and corrosion of the reinforcing steel. Because the penetration of these substances is inevitable, proper concrete cover over the reinforcing is critical to achieve long life.

Conventional reinforced concrete is designed to crack in tension; consequently, the reinforcement that coincides with these cracks may be exposed to corrosive agents soon after the

[18] Roy, Salil, K. and Northwood, Derek, O. "Admixtures to Reduce the Permeability of Concrete" SP 170-13 p. 269

[19] Dr. Lar, PhD thesis (incomplete footnote at this time)

structure is put into service. Alternatively, structures built with prestressed concrete will have no cracks transverse to the direction to prestress under service conditions. Temporary crack widths during construction should be less than 0.30mm provided the cracks are crossed by reinforcing steel and will be relieved from stress in service.

Prestressed concrete requires supplemental steel reinforcement to control the formation and propagation of numerous crack mechanisms. This is provided by the use of plain steel reinforcement for stirrups, spiral coils, bolsters, T-headed bars, anchor dowels, stitch bolts and transverse reinforcement.

Lightweight concrete bridges and other exposed structures have a proven history of long-term performance.[20] Compared to structures using normal weight aggregates, LWC will have greater resistance to microcracking because of their lower modulus of elasticity, lower coefficient of thermal expansion/contraction and strain compatibility at the aggregate-cement matrix interface.[21] Additionally, prestressed lightweight concrete structures are capable of providing good energy absorption.

The designer of MOB shall use the following as a minimum basis for design to provide durable performance for the design life of the MOB.

> In the Serviceability State, concrete that is fully submerged should have crack widths less than 0.25mm. Membrane shear cracks and transverse cracks in the hull shall be less than 0.15mm in the splash zone. Through cracks in external bulkheads and ballasted tank walls should be less than 0.10mm.[22]
>
> The hull shall be prestressed longitudinal and transversely (if necessary) so as to prevent the development of repeated cyclic tension under the waves in the serviceability limit state.
>
> Pontoon and other portions of the hull should be sloped and scuppers should be adequate to prevent ponding of water.
>
> Minimum reinforcement in both directions, and on both faces should be provided in order that a crack, which opens for any reason, will be restrained by rebar below yield stress. The potential tension zone is defined as (c + 7) where c = cover thickness and = diameter of rebar transverse to the potential crack. The area of required reinforcing is $A_S = A_C f_{Ct} / f_y$, where A_C is the area of concrete in a unit length of thickness equal to the potential tension zone, f_{Ct} is the flexural tensile strength of the concrete at age 7 days, and f_y is the yield strength of the reinforcement. For example, for concrete with f'_C of 60MPa at 28 days and f_y = 400 MPa, this results in a requirement of 0.8% reinforcement in the tension zone.

[20] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed structures" page 1, Draft October 1996.

[21] Vaysburd, A.M. 1992. Durability of Lightweight Concrete and its Connections with the Composition of Concrete, Design, and Construction Methods. Proceedings of the International Symposium on Performance of Lightweight Concrete, ACI SP-136 pp. 295-318

[22] Gerwick, Ben, C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993, p. 141

Adequate prestressing and/or reinforcing steel shall be provided to effectively resist in-plane (membrane) shear. This shall take into account the reduction in efficiency of orthogonal reinforcement for resisting diagonal cracking.

Fatigue of prestressed concrete under cyclic loading of waves can be effectively resisted by keeping the stresses in the serviceability Limit State below $0.5f'_C$ compression and zero tension. Fatigue must be considered since it has been well established both by tests and experience in the North Sea that fatigue for concrete members, especially those submerged in water, under repeated waves is a serious problem. [23]

Through-thickness reinforcement shall be provided in all elements subjected to high compressive stresses in the extreme Limit State in order to provide confinement. Such through thickness reinforcement is also required in the sides of hulls to resist transverse (out-of-plane) shear and impact.

Adequate prestressing and/or reinforcing steel shall be provided to effectively resist in-plane (membrane) shear. This shall take into account the reduction in efficiency of orthogonal reinforcement for resisting diagonal cracking.

10.3.1.6 Materials for Construction

MOB design life can be accomplished by using lightweight aggregate and modified density concrete, and implementing sound construction practices and quality control.[24,25,26,27,28] The following considerations for the design, criteria development and specifications are provided.

[23] The Lacy V. Murrow floating bridge in Lake Washington developed cracks over the years that led (or contributed) to its sinking. Now the Evergreen Point Bridge, also in Lake Washington, has started to develop similar cracks at the same age. It is currently undergoing a major retrofit.

[24] Neville, A., "Point of View: Is Strength Enough? Maintenance and Durability of Structures" Concrete International, November 1997 pp.52-56

[25] Lamond, J. F., "Designing for Durability" Concrete International, November 1997 pp. 34-36

[26] Mehta, P. K. "Point of View, Durability – Critical Issues for the Future", Concrete International, July 1997, pp. 27-33.

[27] Gerwick, Ben, C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993 pp. 137-177

[28] The Japan Society of Civil Engineers, Guidelines for Durability Design of Concrete Structures (Draft), 1996

Use prestressing and closely spaced reinforcement to prevent and or minimize crack width.
Use lightweight aggregates that are resistant to water absorption, limit absorption to 8%.
Use controlled processing of the aggregates.[29]
Use proper aggregate grading.
Limit size of the coarse aggregate to 20 mm.
Ensure that the concrete has low water permeability.
Use good design and construction practices to minimize thermal gradients during curing.
Restrict maximum temperature of the concrete during curing to 65^{o}C.
Use rounded or chamfered corners to minimize impact damage.
Use 6 to 10% of aluminate in the cement.
Use a SO_3 cement content of less than 0.4% by mass.
Use fly ash, ground blast furnace slag, and microsilica.
Use small diameter reinforcing steel.
Use chemical admixtures and low w/cm ratio to enhance the air-void system, placement, compaction and workability of the concrete mix.
Use adequate quality concrete cover over prestressing steel, 75mm minimum. Cover over conventional reinforcement shall be 50-mm minimum.
At least 50% of the concrete coarse aggregate shall be lightweight. Fine aggregates may be normal weight. Aggregates must be sound and resistant to the mechanisms of deterioration.

The lightweight concrete and the prestressed tendons must remain sufficiently dimensionally stable.

Provide for proper and complete consolidation of the fresh concrete.
Use proper finishing and curing methods.
Seal all joints.
Consider concrete surface protective coatings.
Use qualified and comprehensive inspection to ensure that the construction conforms to the specifications and the intent for a durable structure.
All concrete which will be exposed above water in normal service shall be properly air entrained to prevent freeze-thaw attack.

10.3.1.7 Reinforced Concrete Design Criteria

The design criteria shall be developed from these ABS Guidelines and applicable world-wide industry standards. Note that durability is determined by many factors, especially impermeability, and is not directly related to concrete strength.[30] Draft design criteria based on performance characteristics are presented in section 10.3.2.2.

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[29] Sandvik, M. et. al. "Chloride Permeability of High-Strength Concrete Platforms in the North Sea" CANMET/ACI International Conference on Durability of Concrete, Nice, France, May 1994

[30] Gjorv, O.E. "Steel Corrosion in Concrete Structures Exposed to Norwegian Marine Environment" Concrete International, April 1994 pp. 35-39

10.3.1.8 Material Specifications

The specifications are the link between the vision of the Department of Defense and the construction of the project. They provide a written vehicle between the Navy and Contractor to meet the Navy's needs. To accomplish this objective, the specifications must be easy to understand and to implement, therefore good specifications are fundamental to the project's success. The designer must write the specifications for the project. The specifications will be performance based in their nature.

10.3.2 Criteria for Reinforced Concrete

10.3.2.1 Final Criteria

The final criteria shall be developed using a systems approach. Various tools are available to aid in the prediction of the concrete durability. Examples include trial mixes, durability studies (computer models and laboratory testing) and finite element analysis for predicting thermal stresses. Extreme caution must be exercised in accepting claims and test data provided by material suppliers. A full-scale mock up of critical sections should be made in order to verify the analytical result. It is important to observe and measure any cracking due to thermal strains, shrinkage strains, and prestressing.

10.3.2.2 Draft Criteria

Table 10.1 presents draft design criteria for MOB. Industry standards contained in the draft criteria include American Concrete Institute (ACI), Norwegian Standard (NS), American Standards for Testing and Materials (ASTM). CEB, New Zealand Concrete Structures Standard (NZS).

Table 10.1 Draft Design Criteria for MOB

Property	Criteria	Relevant Specifications
Concrete Durability	w/(c+m) splash zone < 0.35 w/(c+m) atm. & submerged < 0.37 Min. (c+m) (splash zone) 400 kg/m^3 (675 lb./cu yd) Min. cement content (atmospheric and submerged) 350 kg/m^3 (590 lb./cu yd)	ACI 318, ACI 605, ACI 306 NS 3473 E CEB Bulletin 238 pr EN206 BSI Document 97/104685 NZS 3101:1995
Ec	Ec: 20-25 Gpa (2900-3600 psi) LWA.	ASTM C469
Constructibility	Minimum bleeding and segregation Consistent quality and constituents Control of batching and distribution Slump 200-250 mm with plasticizers	ACI 211.3 ACI 143
Cement	Blended cement	ASTM C 150, ASTM C 595, ASTM C 845. Japanese Belite-Rich Type A HS 65 Norcem
Mixing Water	Chloride ions < 650 ppm Sulfate ions < 1000 ppm No oil, No nitrates	
Aggregates	Lightweight Aggregates Normal weight Fine Aggregates Resistance to abrasion and degradation Resistance to disintegration by sulfates Particle shape and surface texture Grading Bulk unit weight Absorption and surface moisture Aggregate constituents Resistance to alkali-aggregate reactivity	ASTM C 330 ASTM C 33 ASTM C 131, C 535, C 779 ASTM C 88 ASTM C 295, D 3398 ASTM C 117, C 136 ASTM C 29 ASTM C 70, C 127, C 128, C 566 ASTM C 40, C 87, C 117, C 123, C 142, C 295 ASTM C 227, C 289, C 295, C 342, C 586
Chemical Admixtures	High range and normal plasticizers as required to obtain workability and consolidation. Entrained air content 3-7%	ASTM C 494 ASTM C 260

Mineral Admixtures	Silica fume 6-8% by mass of cement Fly ash Type C, F or N (optimum dosage to be established by experimental program).	ASTM C 1240 ASTM C 618 ASTM C 989
Compressive Strength	55-70 MPa (7900-10,000 psi) lightweight aggregate concrete (LWA)	ASTM C 469
Tensile Strength	Determined by the structural design.	ASTM C496 (splitting) ASTM C78 (flexural)
Thermal Stresses	Maximum concrete curing temperature 60 C Maximum gradient 20 C per 300 mm (36 F per 12 in.)	ACI Committee 207
Chloride Contamination	Not to exceed 0.9 kilograms per cubic meter at 75mm depth after 40 years (1.5 pounds per cubic yard)	Laboratory tests using same materials proposed for construction and Fick's Second Law of Diffusion
Conventional Curing	Continuous moist curing for 7 days	ACI 308 ACI 305R
Steam Curing	60^{o}C maximum	
Control Surface Cracks width	During construction, 0.30mm (if crack is crossed by rebar) Serviceability state: 0.15mm in splash and atmospheric zones, 0.20mm submerged.	ACI 318 Norwegian Standard NS 3473 E. CEB Bulletin 238 - New Approach to Durability Design
Fatigue Resistance	Unless justified by detailed analysis, place limits under serviceability limit state of 0.5 f'c compression , zero tension	
Freeze/Thaw Resistance	*Note: Conventional freeze thaw tests and methods are inadequate. Proposed criteria must be carefully prepared and evaluated with regard to research by the Canadian Dept. of Minerals and US Waterways Experiment tests at Treat Island, Marine*	
Resistance to Abrasion	For lightweight aggregate, use of Silica Fume is essential.	ASTM C 779

Depth of Carbonation	Not to exceed 1mm per year	Indicator test pH < 10 ASTM STP 169A
Resistance to Impact	*Under evaluation*	
Prestressing steel	7-wire strands of cold drawn wire, stress-relieved, low relaxation. 75mm concrete cover minimum. Ducts. Corrugated plastic ducts, not less than 0.25mm thickness, and anchorage caps, Grout for tendons. To be neat cement grout, with up to 15% fly ash allowed, and including a non-bleed thixotrophic admixture.	NS 3420, chapter L45
Supplemental steel rebar	Grade 400 to grade 500 Mpa weldable reinforcing steel. Fusion Bonded Epoxy Coated Steel Reinforcement for all stirrups, confinement, bolsters, T-headed anchors, and cast-in place fasteners located in the tidal zone and above.	ASTM A 934 A
Hybrid Structures	Structural Steel and Reinforced Concrete Hybrid Structures are allowed	NS 3476
Quality Control	Third Party Continuous Q.A. Inspection and Documentation	ACI 318

10.3.3 Reinforced Concrete for MOB Construction

10.3.3.1 Scope

To obtain durable concrete, one must carefully consider the materials to be used in the construction. Material characteristics that affect concrete permeability are of particular importance. Prestressed, structural lightweight concrete (LCW) and modified density concrete is the recommended material for the construction of MOB. The scope includes hybrid design concepts employing LCW in combination with structural steel construction.

10.3.3.2 Performance History

LWC has a good performance history of producing high-quality concrete for the marine environment.[31, 32] Concrete has been used successfully by the United States for maritime ship and barge construction since 1918 and can perform on an equal basis with comparable steel vessels.[33] "Concrete ships constructed during World War II are still in service and showing little deterioration."[34] "High strength, prestressed, lightweight concrete also offers excellent durability and energy absorption -- two important considerations in harsh environments".[35] Numerous large prestressed LWC marine structures have proven to be durable, including Hibernia, CIDS offshore platform in the Beafort Sea, the Coronado Bridge in San Diego, and the San Francisco-Oakland bay bridge roadway deck. Several platforms in the North Sea have successfully utilised lightweight or modified density concrete.

10.3.3.3 Cementitious Materials

The cementitious materials include all materials that chemically bind to form the hardened paste including; cement, fly ash, silica fume and rice husk ash. The cementitious materials must be selected to resist the deterioration mechanisms. Portland cements should comply with ASTM C 150, ASTM C 595, and ASTM C 845. The optimum proportions of ground granulated blast-furnace slag, are 70% and 30% cement. The grind (Blaine fineness) of ground granulated blast furnace slag should be less than 380,000mm^2 per/g. (3800 $^{cm2}/_{g}$). Other cements may be considered to satisfy the performance and durability requirements such as Japanese Belite-Rich or HS 65 Norcem. The use of 5 percent silica fume was used to construct Washington State floating bridges.

10.3.3.4 Water-Cementitious Materials Ratio [w/(c+m)]

Durability is directly related to the [w/(c+m)]. It is defined as the ratio of water used to the amount of cementitious materials used in the fresh concrete mixture. The total water content should include the free water on the aggregate but normally excludes that absorbed by the aggregate prior to initial set. Criteria for w/(c+m)] during full-scale production should be established based on the results of the durability study and industry standards.[36] The cementitious materials include all materials that chemically bind to form the hardened paste.

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[31] Holm, T.A. 1980 Performance of Structural Lightweight Concrete in a Marine Environment. ACI Publication, SP, 1-15.

[32] The Federation International de la Precontrainte (FIP) report, State-of-the-Art Report: Lightweight Aggregate Concrete for Marine Structures.

[33] Technical Division of Concrete Control Subsection. 1944. History of the Concrete Ship and Barge Program 1941-1944. U.S. Maritime Commission

[34] Barge-Like Structures ACI Committee Report 357.2R-88 p29. 1988

[35] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Draft October 1996.

[36] ACI 318R-89

10.3.3.5 Aggregates

The selection of an aggregate source that has a proven record for producing stable and durable materials is essential to meeting the durability requirements for MOB. Lightweight aggregates are typically, processed natural materials yielding concrete with a unit density of less than 120 pcf (1920 kg/m^3). Examples include; expanded clay and shale. Use of modified density concrete with a fraction of hard rock aggregate may be considered in order to obtain higher modulus and compressive strength. The ultimate strength and durability properties are highly affected by the characteristics of the aggregate. Important characteristics include; water absorption, water content, uniform strength, stiffness, grading, degree of burning, unit weight, size, surface texture, creep, shrinkage, pore structure, contaminates and manufacturing processes.[37, 38] The compressive strength of the hardened concrete is often limited to the properties of the lightweight aggregate selected. Compressive strengths of 9000 psi (62 MPa) and greater can be reliably produced. ASTM C 330 does not provide sufficient restrictions nor guidelines for lightweight aggregates for marine applications.

It is critically important to select the highest quality aggregate available in order to obtain high performance concrete. Although high-quality lightweight aggregates have been obtained from the U.S. for the Hibernia platform and for major bridges, from Japan for the CIDS, and from Germany for the Troll platform in the North Sea, there are no standards yet established. Critical properties are compression strength, modulus, creep and shrinkage, water absorption and abrasion resistance. In exposures subject to freezing, a maximum of 8% moisture absorption should be required. The lightweight aggregates should not be pre-soaked. Reference is made to research on high-performance lightweight aggregates by the Petroleum Operators' Research Association, as published in the ACI journal. The code and standards requirements for resistance to alkali-aggregate reactivity are not proving adequate for extended marine service beyond 30-50 years. Therefore, the addition of silica in the form of Fly ash and silica fume is important. Petrographic analyses of natural aggregates, both coarse and fine, are recommended.

10.3.3.6 Chemical Admixtures

Admixtures are commonly used to modify the properties of the fresh and hardened concrete. Care should be taken to understand their side-effects and interactions. Their effects are often time and temperature related.[39] Their use with lightweight aggregate concrete may have different results from those experienced with normal weight materials. Trial batches and sensitivity studies are recommended to optimise the selection and dosages used.

[37] ACI 221R

[38] ACI 213R

[39] ACI 212.3R, Chemical Admixtures for Concrete

10.3.3.7 Steel Reinforcement (to be written)

10.3.3.8 Prestressed Reinforcement (to be written)

The use of a zero tension under service load criterion should be considered.

10.3.3.9 Fusion-Bonded Epoxy-Coated Steel Reinforcement

The U.S. Navy in co-operation with industry has developed an ASTM Standard for prefabricated fusion-bonded epoxy coated steel reinforcement for use in severe environments.[40] The U.S. Navy has used this technology successfully to construct several piers and the Admiral Clarey Bridge in Pearl Harbor, Hawaii in 1997-98.

The decision to use epoxy-coated rebar in new Navy construction was based on extensive evaluations that began in 1984. The Naval Civil Engineering Laboratory conducted long-term field evaluations at Key West, Florida to rank the relative performance of popular corrosion control methods. Damage-free epoxy-coated rebar performed best.[41] In contrast to these results, some State Highway Agencies experienced a few projects were epoxy-coated rebar performed poorly. Consequently, the Navy identified the failure mechanisms in current practices and drafted new criteria in cooperation with industry experts. The American Society of Testing and Materials (ASTM) used the Navy's draft specifications as a basis for the development of ASTM A 934/A 934M published in July 1995, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." In February 1998, the NAVFAC Criteria Office published, for the first time, a definitive guide for Marine Concrete, NFGS 03311. Included is a requirement to use prefabricated epoxy-coated reinforcing steel according to ASTM A 934/A 934M .

10.3.4 Considerations for Long-Term Materials Performance

10.3.4.1 Scope

This section presents an introduction to some of the performance relationships that should be considered. To accomplish the durability objective for MOB, emphasis must be placed on understanding the relationships between the environmental conditions, physical loading, deterioration mechanisms and material properties. There are many significant factors involved in these relationships that will ultimately effect the durability of the structure.[42] It is expected that the designer will review the literature and use the information to aid in formulating the construction criteria.

10.3.4.2 Abrasion Resistance

[40] American Society for Testing and Materials, Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars, ASTM Designation: A 934/A 934M-97

[41] Concrete Reinforcing Steel Institute Research Series 2, July 1994.

[42] ACI Committee Report 213B "State-of-the-art Report on Structural Lightweight Aggregate Concrete for Bridges and Other Exposed Structures" Chapter 4 Properties of Lightweight Concrete. Draft October 1996.

Abrasion of the concrete surface due to berthing, docking operations, mechanical equipment, cables or ice scouring may wear away the concrete surface. Reinforcement must be placed with sufficient concrete cover to avoid risk of damage or failure. Strength of the cement matrix, bond to the aggregate and hardness of the aggregate must be considered. Considerable data is available because of work done on LWC marine arctic facilities for the oil and gas industry. Relative performance of candidate materials may be evaluated using ASTM C 779.

10.3.4.3 Creep

Excessive creep of the LWC under load can result in cracks and excessive deformation of the structural element. The creep properties of the specified LWC must be determined by laboratory testing since each mixture has unique properties. Creep characteristics are a function of many variables, including the aggregate, mix design, moisture content of the aggregate, curing method, and the ambient humidity and temperature of the exposure site. Creep is accelerated during steam curing and will result in loss of prestress stain.[43]

10.3.4.4 Durability Study

Computer models are being developed by various organizations world wide that are striving to predict the service life of reinforced concrete in the marine environment.[44,45,46,47] Appropriate models should be identified and evaluated for use. The purpose is to use the model to determine what combinations of material properties are needed to produce a prestressed LWC that will perform as required.

10.3.4.5 Crack Control

The matter of crack width control and its impact on corrosion and durability is the most controversial matter in concrete design and construction today. The problem is the exceeding complexity of cracking, the inability to accurately complete and to measure crack widths, the seemingly chaotic relationship between these and the onset of corrosion. The Norwegian Code NS3473 represents one of the most conservative codes for the amount of reinforcing steel required to control crack width. Excessive use of steel may be counter-productive and expensive.

Variables that affect Crack Widths

[43] Gerwick, Ben, C. "Construction of Prestressed Concrete Structures" Second Edition, Wiley Professional Series, 1993, p. 23

[44] A. Yamamoto, K. Motohashi, S. Misra and T. Tsutsumi, Proposed Durability Design for RC Marine Structures, Concrete Under Severe Conditions Environment and Loading, Volume 1, pp. 544-553, CONSEC '95, 1995

[45] Yokozeki, K., Motohashi, K., Okada, K., and Tsutsumi, T., A Rational Model to Predict the Service Life of RC Structures in Marine Environment SP 170-40 pp.778-799

[46] Collins, F.G., and Grace, W.R., Specifications and Testing for Corrosion Durability of Marine Concrete: the Australian Perspective, SP 170-39 pp. 758-777

[47] Maage, M., Helland, S. and Carlsen, J.E., Service Life Prediction of Marine Structures, SP 170-37 pp.724-743

Temperature
Relative humidity and/or degree of water saturation
Time (age)
Orientation and amount of reinforcement
Cover thickness
Rate of loading
Percent of reinforcement transverse to cracks
Size of bars and spacing
Number of cycles of loading, thermal extremes and wetting-drying

Variables that affect Corrosion Rate of Reinforcing Steel
Permeability of concrete cover to oxygen
Cover thickness
Moisture and relative humidity, degree of saturation
Concentration and size of bars
Stress in the bars

10.3.4.6 Impact Resistance: Impact can best be resisted by use of headed rebars or stirrups in amount of 25% of the through thickness area, in zones of potential impact. (this section hasn't been written yet)

10.3.4.7 Shrinkage: With proper curing, shrinkage strain can be limited to about 400 microstains, ie. $400x10^{-6}$ (this section hasn't been written yet)

10.3.4.8 Corrosion Resistance (this section hasn't been written yet)

10.3.4.9 Fatigue Resistance (this section hasn't been written yet)

10.3.4.10 Fire Resistance Fire resistance is important in interior areas where equipment is operating. High strength lightweight concrete will not provide adequate fire resistance but will spall, probably explosively. Therefore a separate insulating layer of vermiculite or other fire resistant insulating concrete as necessary to obtain the desired resistance to fire.

10.3.5 Concrete Materials for Aircraft Traffic Surfaces

10.3.5.1 Scope

Concrete, steel, or a combination of the two may be considered for the MOB aircraft traffic areas. All of the general requirements for concrete, previously stated apply to aircraft traffic surfaces. This section provides supplemental requirements and considerations that are unique to Navy aircraft operations.

10.3.5.2 General Considations

Concrete requires periodic maintenance including crack sealing, patching, and re-sealing of expansion joints. In addition, concrete is subject to the buildup of rubber deposits from landing aircraft that must be removed periodically using high pressure water blasting. Repeated high pressure water blasting will abrade the concrete thickness. Ether 75mm cover or a 37mm additional topping of latex-modified morter., You may want to reference the La Guardia Airport Overwater Runways and Taxiways originally built in 1965 or prestressed hard-rock concrete and under continuous heavy air traffic ever since.

Steel requires a non-skid coating similar to that used on aircraft carriers and landing mats (AM2 mat), that must be re-applied periodically. The designer must also consider the following.

Stresses and possible damage to the concrete aircraft deck surface due to the flexibility of an underlying steel structure.
Inspection, maintenance, and repair of concrete
Frequency, cost, and impacts to operational availability.

10.3.5.3 Applicable Documents

Most all documents for concrete pavement design are based on the premise that the substrate is compacted and stable.[48] The MOB platform may be a relatively flexible surface to construct a concrete aircraft deck upon. Criteria developed for MOB aircraft deck surfaces must be based on simulation from laboratory and field-testing of s floating airfield pavements. The following documents are provided for reference because they contain criteria for aircraft traffic surfaces.

MIL-HDBK-1021/1:	Airfield Geometric Design
MIL-HDBK-1021/2:	General Concepts for Airfield Pavement Design
MIL-HDBK-1021/4:	Rigid Pavement Design for Airfields
NAVFAC DM 21.9:	Skid Resistant Runway Surface
MIL-HDBK-1023/1:	Airfield Lighting
MIL-HDBK-1024/1:	Aviation Operational and Support Facilities
NAVFAC DM 21.06:	Airfield Subsurface Drainage and Pavement Design
NAVFAC DM 2.04:	Concrete Structures

[48] MIL-HDBK-1021/4, Rigid Pavement Design for Airfields,

MIL-HDBK-1002/1:	Structural Engineering General Requirements
MIL-HDBK-1002/3:	Steel Structures
NAVFAC P-80:	Criteria for Navy and Marine Corps Shore Installations
ASTM D 2628	
FAA Advisory Circular 150/5320-12C, Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces	

10.3.5.4 Concrete

All loading aspects, including static and dynamic that would influence the integrity of the surface to carry the aircraft loading, shall be considered. The fatigue characteristics shall be determined based on the proposed amount of traffic and the combined effect of critical and non-critical design aircraft. The concrete shall perform as the wearing surface for aircraft traffic and meet the following draft criteria.

Table 10.2 Draft Criteria for Concrete Aircraft Surfaces

Maximum longitudinal slope of 0.5 percent.
Transverse crown with slope no less than 1 percent or greater than 1.5 percent.
Skid resistance within acceptable limits of > 0.7 (M_u number) per U.S. Navy and Federal Aviation Administration (FAA) Advisory Circular AC No. 150/5320-12C dated 3.18.97, Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces.
Roughness index such that resonance frequency of the aircraft is avoided for all anticipated aircraft and landing situations. Acceptable Roughness is a function of wheel spacing, mass of the aircraft, speed, landing gear suspension, and other parameters.
The concrete shall be maintainable according to the Condition Indexes (PCI) and not fall below the value of 70 for the runway and 60 for all other areas subjected to aircraft traffic.
Shall be structurally adequate for all loading conditions including but not limited to aircraft static and dynamic loads, and blast and high temperature exhaust effects.
Shall be resistant to the combined effects of fatigue and differential movements.
Shall be resistant to abrasion of wheel loads and arresting gear.
Concrete reinforcement can be ordinary deformed reinforcing bars, prestressed, and/or post-tensioned reinforcing strand.
Fusion-bonded epoxy-coated steel reinforcement shall not be used in the runway.
Supplemental reinforcement such as synthetic fibers are allowed.
The surface shall not contain any steel fiber reinforcing material
Longitudinal Joints - use Preformed Polychloroprene Elastomeric Joint Seal
Transverse Joints - use Dow Corning 890-SL Self-Leveling Silicone Joint Sealant (or equivalent) with a compatible backer rod as recommended by the manufacturer.
Airfield Marking Paints - as recommended in NAVFAC Guide Specification 02761A.
Aircraft Tiedowns - as recommended in NAVFAC Guide Specification 02762A and MIL-HDBK-1021/4
Arresting Gear Inlays: Use NAVFAC Definitive Steel Plate Design (NAVFAC definitive drawings 1404521 and 1404522)
VSTOL Launch Pads: Use AM-2 Matting
Parking Areas Subject to Jet Aircraft Auxiliary Power Units: Place an inlay of high performance concrete resistant to blast and high temperature exposures. Proprietary materials such as "Set 45" cement used with a lightweight aggregate such as "Solite" has been evaluated by the Naval Facilities Engineering Service Center as having enhanced properties compared to conventional concrete mixtures. Other surfaces resistant to blast and high temperature exposure such as steel plates or AM-2 Matting may be used.

10.3.6 Quality Control

The literature recommends that strict adherence to quality control is necessary to assure that good concrete placement practices are followed. The contract must contain necessary quality control requirements to assure that the repairs are accomplished satisfactorily.

10.3.6.1 Third Party Inspection:

The Navy will hire a third party certified Quality Assurance (Q.A.) firm to assure that the highest quality product is consistently produced for MOB construction. The Q.A. firm must have qualified experience and a full time licensed engineer. Daily Q.A. documentation must be submitted to the Contract Officer for review.

10.3.6.2 Demonstration:

The contractor shall successfully demonstrate to the Navy a successful method of forming, placing, prestressing, and curing the concrete prior to full-scale production. All appropriate parties should witness and approve the materials, procedures and quality control program. The demonstration will set the quality standard for the full-scale production.

10.3.6.3 Laboratory Testing

10.3.6.4 Field Controls

Aggregate
Mixing
Placement
Unit Weights
Air Content
Finishing
Curing

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PART C

PATCH REPAIRS

This section contains guidelines, specifications, and case studies for marine concrete repair above and below the waterline.

CHAPTER 7

NFESC SPECIFICATIONS FOR MARINE CONCRETE REPAIR

NFESC

SPECIFICATIONS FOR:

Marine Concrete Repair

For:

Top deck patch repair
Under deck hand pack patch repair
Drip edge for penetrations
Crack repair
Joint repair

MARINE CONCRETE REPAIR
SITE DEMOLITION

1. GENERAL

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to in the text by the basic designation only.

AMERICAN NATIONAL STANDARDS INSTITUTE (ANSI)
CODE OF FEDERAL REGULATIONS (CFR)

1.2 GENERAL REQUIREMENTS

Do not begin demolition until authorization is received from the Contracting Officer. Continuously remove rubbish and debris from the project site; do not allow accumulations on the structure deck. Store materials that cannot be removed daily in areas specified by the Contracting Officer. Contain all materials from entering the harbor waters.

1.3 SUBMITTALS

Submit the following documentation to the Contracting Officer prior to receiving authorization to proceed with demolition.

1.3.1 Statements

a. Demolition plan
b. Notification of demolition and renovation

Submit proposed demolition and removal procedures to the Contracting Officer for approval before work is started.

1.3.1.1 Required Data

Demolition plan shall include procedures for coordination with other work in progress, a detailed description of methods and equipment to be used for each operation and of the sequence of operations.

1.4 REGULATORY AND SAFETY REQUIREMENTS

Comply with federal, state, and local hauling and disposal regulations. In addition to the requirements of the "Contract Clauses," safety requirements shall conform with ANSI A10.6, "Demolition Operations - Safety Requirements."

1.4.1 Notifications

Furnish timely notification of demolition and renovation projects to Federal, State, regional, and local authorities in accordance with 40 CFR 61-SUBPART M, if required. Notify the local air pollution control district/agency and the Contracting Officer in writing 10 days prior to the commencement of work in accordance with 40 CFR 61-SUBPART M.

1.5 DUST AND DEBRIS CONTROL

Prevent the spread of dust and debris and avoid the creation of a hazard or nuisance in the surrounding area. Do not use water if it results in hazardous or objectionable conditions such as, but not limited to, pollution or runoff into the harbor waters. Minimize the application of water to that required for dust control. Prevent unpermitted discharges into the storm sewer, soil and harbor. Water washdown of the areas is not allowed. Prevent all debris concrete cutting, chipping, demolition, and repair from entering the harbor waters. The contractor must retrieve any material that enters harbor waters.

1.6 PROTECTION

1.6.1 Traffic Control Signs

Where pedestrian and driver safety is endangered in the area of removal work, use traffic barricades with flashing lights. Notify the Contracting Officer prior to beginning such work.

1.6.2 Existing Work

Protect existing work that is to remain in place, be reused, or remain the property of the Government. Repair items which are to remain or which are to be salvaged that are damaged during performance of the work to their original condition, or replace with new. Do not overload structural elements. Additional structural supports and reinforcement must have Contracting Officer approval.

1.6.3 Facilities

Protect electrical and mechanical services, utilities, and facilities. Where removal of existing utilities and pavement is specified or indicated, provide approved barricades, temporary covering of exposed areas, and temporary services or connections for electrical and mechanical utilities.

1.7 BURNING

Burning will not be permitted.

2. EXECUTION

2.1 EXISTING FACILITIES TO BE REMOVED

2.1.1 Concrete

Break out the concrete and prepare repair surface as detailed in the Contract Drawings and the Specifications for REPAIRS.

2.1.2 Demolition

Prior to the start of demolition or crack repairs, the areas to be demolished or repaired shall be marked out and jointly inspected by the Contractor, Contracting Officer and quantities estimated as specified under the Specifications for DECK REPAIRS and SILICONE CRACK REPAIR. Abandoned pipelines and conduits may be removed at the discretion of the contractor to facilitate concrete repairs.

2.2 TITLE TO MATERIALS

Except where specified in other sections, all materials and equipment removed, and not reused, shall become the property of the Contractor and shall be removed from Government property. Title to materials resulting from demolition, and materials and equipment to be removed, is vested in the Contractor upon approval by the Contracting Officer of the Contractor’s demolition and removal procedures, and authorization by the Contracting Officer to begin demolition. The Government will not be responsible for the condition of, loss of, or damage to, such property after contract award. Materials and equipment shall not be viewed by prospective purchasers or sold on the site.

2.3 CLEANUP

2.3.1 Debris and Rubbish

Remove and transport debris and rubbish in a manner that will prevent spillage into the ocean, harbor or bay waters, on streets, or adjacent areas. Clean up spillage from the structure and adjacent areas daily.

CONCRETE REPAIRS
DECK REPAIRS

1. GENERAL

This specification covers the use of prepackaged cementitious concrete repair materials and procedures for making partial-depth repairs to Structure.

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

AASHTO T 2771989	Standard Method of Testing for Rapid Determination of the Chloride Permeability of Concrete

AMERICAN CONCRETE INSTITUTE

ACI 301 (1994)	Structural Concrete for Buildings

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM C 309 (1994)	Liquid Membrane-Forming Compounds for Curing Concrete
ASTM A 615	1993 Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
ASTM C 33	1993 Concrete Aggregates
ASTM C 109	1991 Standard Method for Compressive Strength of Hydraulic Cement Mortars
ASTM C 157	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
ASTM C 490	Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete
ASTM C 496	1990 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
ASTM C 882	1991 Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete (modified for cementitious material)
ASTM C884	1987 Test Method for Thermal Compatibility Between Concrete and an Epoxy-Resin Overlay

1.2 DESCRIPTION OF WORK

Concrete repair work must be accomplished prior to installing the cathodic protection system and the application of composite upgrade materials. The concrete repair work consists of several parts: A) Partial-depth repairs of deteriorated concrete on the top and bottom of the structure deck. B) Removal of steel crane rail and placement of concrete into the resulting cavity. C) Removal of sound concrete that protrudes from the under deck surfaces. These areas are identified as existing "built-up" repairs. Removal of the built-up areas may result in a cavity, which must be repaired with concrete to establish a suitable surface profile flush with the adjacent surfaces. D) Removal of sound concrete along some of the construction joints on the underside of the deck to establish a suitable surface profile flush with the adjacent surfaces. E) Installation of a drip edge at existing holes on the underside of the deck to direct water away from the concrete surface. F) Sealing construction joints and cracks on the top of the deck to prevent water from wetting the underside of the deck. The repair work shall proceed by removing concrete from the areas identified by the contracting officer using approved methods identified in the Contract Drawings and herein, cleaning the area by abrasive blasting, placing an approved bonding agent, placing an approved repair material, finishing and texturing, curing, and, finally, sealing joints and saw overcuts.

1.3 LOCATION

The Contracting Officer will designate the locations and boundaries of each repair area with the Contractor. The Contractor will remove all unsound concrete and expose the rebar as necessary based on the repair criteria so that no visible corrosion is evident beyond normal "mill scale." Refer to Contract Drawings.

1.4 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with concrete repair. Some laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests with an approved test laboratory well in advance to avoid delays in the concrete repair work.

1.4.1 Manufacturer's Catalog Data/Instructions

a. Cementitious Repair Material
b. Curing Compounds

1.4.2 Laboratory Test Results and Verification

The Contractor will submit to the Contracting Officer test results from an approved concrete laboratory showing that the repair material meets or exceeds the Navy's specifications on shrinkage and strength. Some laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

1.4.3 Batch Samples

When requested by the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 2% of the total material used on this job.

1.5 DELIVERY, STORAGE, AND HANDLING

Inspect materials delivered to site for damage, unload and store with a minimum of handling. Deliver cementitious repair material components and aggregate materials in original sealed containers and store in dry covered areas at temperatures below 100 F.

1.6 WEATHER LIMITATIONS

Halt work when the weather conditions are inclement and detrimentally affect the quality of patching concrete. Windy conditions and rain will affect the concrete curing. Apply patching materials only when the atmospheric and surface temperature ranges are suitable for the specified material. Halt work if the temperature is below 40 F (4 C). Follow manufacturer's instructions for weather conditions and temperature ranges. Patches placed during adverse weather conditions may have to be removed and replaced.

1.7 EQUIPMENT

Use a container recommended by the manufacturer as the mixing vessel. Use equipment specified by repair material manufacturer for field mixing, transporting and consolidation of cementitious repair materials.

1.8 QUALITY ASSURANCE

A Technical Representative of the manufacturer of the cementitious repair material being used shall be present during the start of repair work. The Technical Representative shall inspect and approve the surface preparation and observe the initial application. The Technical Representative may demonstrate and instruct the Contractor on proper procedures.

A written report shall be submitted to the Contracting Officer outlining the Technical Representative's observations and suggestions, including but not limited to recommendations regarding mixing and placement procedures, and equipment used for mixing, placement, consolidation, and curing.

Mixing and handling instructions shall be available at the site at all times during the repair operations.

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person certified by the material manufacturer who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the material manufacturer.

2. MATERIALS

2.1 MATERIAL SPECIFICATIONS

The materials used shall meet the requirements of the following specifications as well as other Contracting Officer approved Proprietary repair materials:

AASHTO M-80 & M-6	Aggregate
AASHTO M-148	Curing compound
AASHTO M-194	Concrete admixtures

2.1.1 Cementitious Patch Material

The product shall be prepackaged by the manufacturer with premeasured, properly proportioned components. It shall be suitable for the hand-packed repair method (see Contract Drawings) and shall have the following properties:

a. Minimum pot life of 30 minutes at 75 F.

b. Bond strength per ASTM C 882 modified for cementitious material at 28 days: 2,200 psi minimum.

c. Maximum permeability of 1,000 coulombs per AASHTO T 277.

d. Drying shrinkage: Specimens shall be prepared per ASTM C 157 as modified to use molds per ASTM C 490 (3x3x11.25 inches) with a 10-inch gauge length. During the first 7 days the molded specimen shall be covered with a water-saturated rug or burlap. After the 7 day wet curing period, the mold shall be removed and the specimen cured for an additional 28 days at 46 to 54% relative humidity at 70 to 76 F. The ultimate shrinkage to be reported is that value measured at the end of the 35th day. Allowable shrinkage shall not exceed 0.05%.

e. Minimum compressive strength per ASTM C 109 modified for cementitious material shall be 3,000 psi @ 3 days.

f. The water to cementitious ratio shall not exceed 0.40.

2.1.2 Aggregate

If aggregate is added to the prebagged mixture, then all tests for acceptance criteria per Section 2.1.1 shall be conducted with the added aggregate. Aggregate added to the repair material, if allowed by the manufacturer, shall be 3/8-inch minus, clean, well graded, saturated surface dry material, having low absorption and high density, and conform to ASTM C 33. Aggregate must be approved for use by the Contracting Officer.

2.1.3 Reinforcing Bars

ACI 301 unless specified otherwise. ASTM A 615, Grade 60 bars.

2.1.4 Laboratory tests per Section 2.1.1 shall be submitted to the Contracting Officer for approval before concrete repair can proceed. Laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

3. EQUIPMENT

3.1 GENERAL

The Contractor shall furnish and maintain such equipment as necessary to complete the work in accordance with the specifications.

3.2 CONCRETE SAW

The concrete saw shall be equipped with a diamond blade(s) or approved equal. The saw shall be capable of sawing concrete to the specified depth without damaging the surrounding concrete. Depth of cut shall be adjusted so as to avoid cutting the existing steel reinforcement.

3.3 CONCRETE REMOVAL EQUIPMENT

The Contractor shall provide equipment capable of removing the deteriorated concrete in the repair area to the depth required without damaging the sound concrete surrounding or below the repair. The Contractor shall provide the necessary means to assure that no concrete debris or slurry water enters the harbor waters, refer to SITE DEMOLITION Specifications.

3.3.1 Pneumatic Jackhammers

Jackhammers heavier than 15 pounds (6.8 kg) shall not be permitted.

3.3.2 Abrasive Blasting or Mechanical Scarification

Abrasive blasting or mechanical scarification shall be capable of removing all contaminants and loose particles from the surface of the steel reinforcement and concrete in the repair area. The equipment shall be fitted with suitable traps, filters, drip pans, or other devices to prevent oil, fuel, grease, or other undesirable matter from being deposited on the cleaned surface and the harbor waters.

3.3.3 Brooms, Shovels

Stiff-bristled brushes shall be used to apply the bonding agent. Shovels may be used to place the repair materials, if appropriate.

3.4 FINISHING AND FLOATING EQUIPMENT AND STRAIGHTEDGES

The finishing and floating equipment shall be capable of consolidating and floating the concrete. A dense, homogenous repair must be produced and finished to the same surface slope as the existing concrete slab.

3.4.1 Pressure Hand Sprayer for Membrane-Curing Compounds

The pressure sprayer for membrane-curing compounds shall be capable of providing a uniform, even coating of the compound over the surface of the repair. Manually operated spray equipment may be used.

4. CONSTRUCTION METHOD

4.1 DETERMINATION OF REPAIR AREAS

The Contracting Officer shall determine areas to be repaired by using a hammer or other techniques to determine the extent of the unsound concrete. The Contracting Officer shall mark the boundaries of the repair area. Large areas such as the rail slot may use flowable repair materials while small areas and all areas below deck shall be repaired by the dry-pack method. Holes through the deck will be either filled with low shrinkage concrete or lined with a drip edge according to the Contract Drawings. All previous built-up repairs under the deck that interfere with areas to be structurally upgraded must be modified to achieve a compatible surface profile with the adjacent concrete surfaces. See the Contract Drawings for details of repairs.

4.2 PREPARATION OF REPAIR AREA

A hand-held 15-lb chipping hammer may be used. All other methods must be approved by the Contracting Officer.

4.2.1 Concrete Removal

The deteriorated material in the repair area shall be removed using the methods specified in this section. A saw cut shall be made around the perimeter of the repair area to provide a vertical face at the edges and sufficient depth for the repair. The saw cut shall have a minimum depth of 1 inch (25 mm). Depth of cut shall be selected to preclude cutting reinforcing steel bars.

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (51 mm) or until sound concrete is exposed.

Remove loose concrete from the designated areas. Inspect the cavity for remaining unsound concrete by tapping with a hammer or steel rod. In areas where tapping indicates unsound concrete, remove additional concrete. Make the entire cavity at least 2 inches deep. Where rebar is exposed remove all corrosion by abrasive blasting or mechanical means to a near white metal condition as per recommendations of patch material manufacturer, prior to installing patch material. Continue to “chase” all corroded steel reinforcement until no corrosion is visible beyond normal “mill scale.” Prepare surfaces by abrasive blasting or mechanical scarification to achieve a uniformly rough surface.

4.2.2 Concrete Removal of Built-up repairs (underside of deck)

Existing form and pump concrete repairs made to the underside of the deck were built up to increase the cover over the steel, resulting in areas that are not flush with the adjacent concrete surface. Typically these built-up areas extend 2 to 3 inches from the original surface. These areas must be cut back, so that they are flush to permit the application of the structural upgrade materials. The contractor shall propose to the Contracting Officer the method by which the concrete will be removed. If a cavity results from the removal of the concrete, then this area must be repaired so that the surface is flush with the adjacent concrete.

4.2.3 Hand-Held Chipping Hammer

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (50 mm) with pneumatic tools until sound concrete is exposed. The maximum size pneumatic hammer shall be 15 pounds (6.85 kg). Pneumatic hammers and chipping tools shall not be operated at an angle exceeding 45 degrees from the vertical. Such tools may be started in the vertical position but must be immediately tilted to a 45-degree operating angle. The removal shall start within the interior of the repair and work outward. Care shall be used to prevent fracture of the sound concrete below the repair area and the surrounding concrete. A minimum 1-inch (25-mm) vertical face (saw cut) on all sides shall be provided. However, adjustments shall be made to avoid cutting any steel rebar. All concrete chips/debris shall be contained and prevented from falling into the harbor waters.

4.3 SURFACE PREPARATION

4.3.1 Concrete

Abrasive blast or mechanically scarify the exposed faces of the concrete to remove all loose particles, oil, dust, cement or slurry residue, paint, and other contaminants. Immediately prior to placing the concrete bonding agent, clean the exposed surfaces by compressed air blasting. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the harbor water.

4.3.2 Steel Reinforcement

Reinforcing steel bar that has lost more than 25% cross-sectional area must be repaired by welding a new segment of rebar of the same diameter to the existing rebar. Corroded or damaged rebar will be identified in the field by the contractor and verified for replacement by the Contracting Officer. The splice will cross the damaged length and the welds made at locations where the existing rebar is in excellent condition without loss of area. New reinforcing steel shall be ASTM A-615 grade 60 and welded in accordance with the Structural Welding Code – Reinforcing Steel (AWS D1.4). The welding surface shall be prepared by power cleaning as per SSPC-SP11. The weld will be a continuous 0.25-inch fillet that is at least 2 inches long. The contractor will remove any concrete that is damaged during the welding process. Abrasive blast or mechanically clean the steel to bright steel no more than 48 hours prior to application of concrete patch material. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the harbor water.

4.4 APPLYING THE BONDING AGENT

Use a bonding agent recommended by the supplier of the repair material. It may consist of neat cement, cement-sand, or latex-cement slurry. Bonding agents must be approved by the Contracting Officer. Apply the bonding agent to a clean surface saturated dry (SSD) concrete substrate and scrub it into the surfaces using a stiff-bristled brush. Bonding agents that contain epoxy will not be allowed.

4.5 PLACING THE REPAIR MATERIAL

Always place materials containing aggregate with a shovel to avoid segregation. Flowable materials may be placed by a bucket or other suitable means.

4.5.1 Proprietary Repair Materials

Place dry pack repair materials according to the method in the Contract Drawing. The application shall be in accordance with manufacturer's recommendations. Special attention shall be paid to pack the material below reinforcing bars and to working the material into the concrete substrate to achieve a sound bond. Use a hard wood dowel to ram the material tightly below and around reinforcing.

4.6 FINISHING REQUIREMENTS

Partial-depth repairs are usually small enough so that a stiff board resting on adjacent sound concrete can be used as a screed. Work the materials toward the perimeter of the patch to establish contact and enhance bonding to the existing slab. Make at least two passes with the screed to ensure a smooth repair surface that is level with the surface of the deck. Care should be taken to not "overwork" the surface.
When practical, match the surface texture of the repair with that of the surrounding deck.

4.7 CURING

Spray apply two coats of a concrete curing compound (ASTM C 309) as soon as the concrete surface has set sufficiently to apply the curing agent without damage. Apply the curing compound at the rate of 150 ft^2/gal (3.7 m^2/L). In addition, repairs to the top deck shall also be moist cured for 7 days by covering with saturated pieces of wet rug or carpet.

4.8 SAW OVERCUTS

The saw cuts extending from the repair area into to the surrounding sound concrete must be filled with epoxy mortar or cement mortar.

4.9 OPENING TO STRUCTURE OPERATIONS

The concrete repairs may be opened to structure operations when a compressive strength of 3,000-psi (21 MPa) has been achieved.

5. QUALITY CONTROL

5.1 CONCRETE REPAIR MATERIALS

Material supplied to the job shall comply with Section 2.1.1 of this specification. Test reports shall be from an independent testing laboratory approved by the Contracting Officer.

5.3 INSPECTION

The Contracting Officer shall check each repaired area for cracks, spalls, popouts, and loss of bond between repaired area and surrounding concrete one week after the repair material was placed. Each repair area will be checked for voids by tapping with a hammer. In addition, they may take one, 1-inch diameter core in each span to verify depth, bonding integrity, and material quality of the concrete repair. The Contractor shall repair the cored site to the same level as required by this specification. Areas found to be defective will be removed and replaced by the Contractor to the satisfaction of the Contracting Officer and the required performance and quality level of this specification.

CONCRETE REPAIR
CRACK SEALANT

This specification applies to the requirements for repairing either static or dynamic concrete cracks by sealing on the top surface of the deck. This specification details both application procedures and material requirement. The sealant must provide a service life for a minimum of 10 years in a marine environment.

1. GENERAL

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM D 412	1997 Standard Test Method for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers - Tension
ASTM D 638	1996 Standard Test Method for Tensile Properties of Plastics
ASTM D 1475	1990 Standard Test Method for Density of Paint, Varnish, Lacquer, and Related Products
ASTM D 2240	1997 Standard Test Method for Rubber Property - Durometer Hardness
ASTM D 5893	1996 Standard Specification for Cold Applied, Single Component, Chemically Curing Silicone Joint Sealant for Portland Cement Concrete Pavements

CODE OF FEDERAL REGULATIONS (CFR)

29 CFR 1910.134	Respiratory Protection
29 CFR 1926.59	Hazard Communication
40 CFR 261	Identification and Listing of Hazardous Waste

1.2 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with crack sealing. The Contractor shall use a silicone sealant. The color of the sealant must be gray.

1.2.1 Instructions

a. Two Part Self-Leveling Silicone Joint Sealant

Submit formulator’s printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per size of crack, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA & 29 CFR 1926.59.

1.2.2 Field Test Reports

a. Tests and Inspections

The contractor will submit reports on tests and inspections as set forth in section 3.3.

1.2.3 Certificates

a. Two Part Self-leveling Silicone Joint Sealant

Certify conformance to the requirements set forth in section 2.1.1.

b. Primer Coat Material

Certify conformance to the requirements set forth in section 2.1.2.

c. Bond Breaker Material

Certify conformance to the requirements set forth in section 2.1.3.

1.2.4 Records

a. Installers Qualification.

The Contractor shall have verifiable and specific experience repairing and sealing concrete cracks and expansion joints utilizing routing equipment, bondbreaker tape, and silicone sealant. Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements for crack sealing with silicone, certified by the material manufacturer to be completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures.

b. Disposal of Material.

All unused material, whether in its cured or uncured state, shall be removed from the job site by Contractor. No material will be allowed to enter harbor waters. Any material that enters harbor waters will be retrieved by the Contractor.

1.2.4 Batch Samples

When requested by the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 2% of the total material used on this job.

1.3 DELIVERY AND STORAGE

Ship sealants and other materials in their original, sealed containers. Materials delivered to site shall be inspected for damage and container opening prior to use. Material delivered in dented, rusty, leaking, or previously opened containers and, in addition, material with an expired shelf life shall be returned to manufacturer. Material shall be unloaded and stored out of sun and weather, preferably in air-conditioned spaces.

1.4 SAFETY

Ensure that employees are trained in the requirements of OSHA &29 CFR 1926.59 and understand the information contained in the MSDS for their protection against toxic and hazardous chemical effects. Follow safety procedures as recommended by manufacturer. Procedures may include employing the use of impervious clothing, gloves, face shields, and other appropriate protective clothing necessary to prevent eye and skin contact with materials.

2. PRODUCTS

2.1 MATERIALS

2.1.1 Silicone Joint Sealants

Silicone joint sealants shall be rapid cure, 100 percent silicone, self-leveling, two-part formulation, and cold applied. Acid cure sealants are not acceptable for use on concrete. Silicone sealant shall be compatible with the surface to which it is applied.

Rapid cure is defined as developing sufficient integrity within 8 hours to accommodate both thermal and/or vertical movements due to traffic loading.

Specific Gravity, as per ASTM D 1475	1.25-1.35
Nonvolatile Content (% minimum)	93
Skin-over Time (minutes, maximum)	20
Joint Elongation (% minimum), as per ASTM D 5893 Section 14 modified – pull rate (2 in./min.) and joint size Joint size = ½” x ½” x 2”	600
Joint Modulus (psi, @ 100% elongation), as per ASTM D 5893 Section 14 modified – pull rate (2 in./min.) and joint size Joint size = ½” x ½” x 2”	3-12

2.1.2. Primer Coat Material

Once the crack has been cut, cleaned and dried, the crack shall be coated with a primer prior to the installation of the bond breaker and sealant. The primer coat material to be used shall be from the same manufacturer as that of the silicone sealant and as recommended by the sealant manufacturer and must be compatible with the concrete, bond breaker, and the sealant.

2.1.3. Bond Breaker Material

A bond breaker material shall be installed prior to installation of the sealant.

2.1.2.1. Purpose of Bond Breaker

a. Maintain minimum and/or maximum depth of sealant.
b. Prevent three (3) sided adhesion of sealant. Bond breaker serves to ensure that the bottom of the sealant is bond free thereby allowing sealant to adhere to the sides of the joint only.

2.1.2.2. Requirements of a Bond Breaker

a. Shall be compatible with sealant or any component of the joint sealant system.
b. No bond or adverse reaction shall occur between the bond breaker and the sealant.

2.1.2.3. Acceptable Types of Bond Breakers

a. Closed-cell expanded polyethylene foam backer rod. Primary use is with new joint construction and uniform remedial joint construction.
b. Bond breaker tape or approved equal. Primary use is with wide, shallow joints.
c. Backing material that is open cell with an impervious skin to prevent adhesion. Primary use is with irregular remedial joint construction.

3. EXECUTION

3.1 CRACK PREPARATION

3.1.1 Locate Rebar and Conduit

All rebar and conduit located a minimum distance of three inches from each side of the crack shall be identified and mapped along the length of the crack. Mapping shall include depth of concrete cover and the exact location of rebar and/or conduit in relation to the crack. Cracks shall be 1/4" wide by 5/8" to 1" deep. The backer rod diameter shall be 3/8" with the sealant thickness of 1/4" (even with the top surface). Joints shall be cleaned and prepped by removing the existing sealant and using an abrasive blast to clean the surface of the joint. The following procedures shall be observed for both cracks and joints.

3.1.2 Surface Cleaning

Cracks shall be routed/cut out to 1/4" width and 5/8" to 1" depth without disturbing or damaging existing rebar and conduit. All dirt, debris, efflorescence, chipped concrete, grease, oil, and other obtrusive material in each crack shall be removed both inside and a minimum of one half inch (1/2") in width on both sides of each crack to be repaired. Cleaning shall be accomplished by a combination of wire brushing, hand tool cleaning, power tool cleaning, compressed air, and aqueous based detergent cleaning. Cleaning utilizing organic solvents is prohibited. All dirt, debris, chipped concrete, grease, oil, and other obtrusive material removed shall be contained and prevented from falling into the harbor water.

3.1.3. Primer Coat

Use a primer recommended by the sealant manufacturer. Only apply the primer following the day's high temperature to avoid off gassing of the concrete and to permit maximum penetration of the primer. Prepare the joint by abrasive blasting and then air blowing the joint. Air compressors used for this purpose must be equipped with traps capable of providing moisture and oil free air. As with any application involving primer, backer material should not be installed until the primer is applied to avoid pooling of primer at the joint wall interface.

For best results, apply the primer with a mist sprayer. (Applying with a clean, lint-free cloth is an alternative, although less desirable method.) Uniformly coat the entire surface primer, being careful not to saturate the substrate. If applied correctly, the substrate will darken in appearance, but there should be no signs of primer run down. Once dry, there should be no visible signs of the primer, only a slight odor. If the primer is over-applied, a white powder will form the substrate surface. If this occurs, the joint/crack must be recleaned and the process repeated.

Allow the primer to dry for approximately 60 minutes. Gently air blow the prepared joint/crack, install the backer material and then install the joint sealant as recommended.

3.1.4 Install Bond breaker

The bond breaker shall be applied to the inner base of the routed out, cleaned, and dry crack. Bond breaker shall be placed flush with inner walls of crack.

3.2 SEALANT APPLICATION

3.2.1 Mixing

Based on ambient temperature, relative humidity, and moisture content in concrete, consult sealant manufacturer and mix silicone sealant components in accordance to their recommendations.

3.2.2 Sealant Installation

Immediately following the day's high temperature and on dry concrete, pour into crack the silicone sealant over the polyethylene bond breaker tape and finish by trowel. Resulting crack repair shall be flush with the surrounding concrete, exhibit complete crack depth penetration, and be free of surface irregularities, air voids, and discontinuities greater than 1/32 inch. A primer shall be used to aid adhesion on either questionable concrete or concrete that contains excess moisture. Primer shall be chemically and mechanically compatible with silicone sealant.

3.2.3 Curing

Within forty-eight hours following application of sealant, sealant shall be tack free and ready for light traffic. If after forty-eight hours, the sealant is tacky or in any form of its uncured state, all uncured material shall be removed by Contractor. New sealant will be reapplied to the crack.

3.3 FINAL INSPECTION

Government shall inspect and verify repairs have been carried out in accordance with the guidelines set forth in sections 3.2.2 and 3.2.3. The contractor shall take one, 1-inch diameter core on each span to verify depth and quality of sealant penetration. The contractor will repair the cored site to the same level as required by this specification.

3.4 FINAL CLEANUP

Following completion of work, remove debris, equipment, and materials from the site. Remove temporary connections to Government furnished services. Restore existing facilities in and around the work areas to their original condition.

GENERAL NOTES:

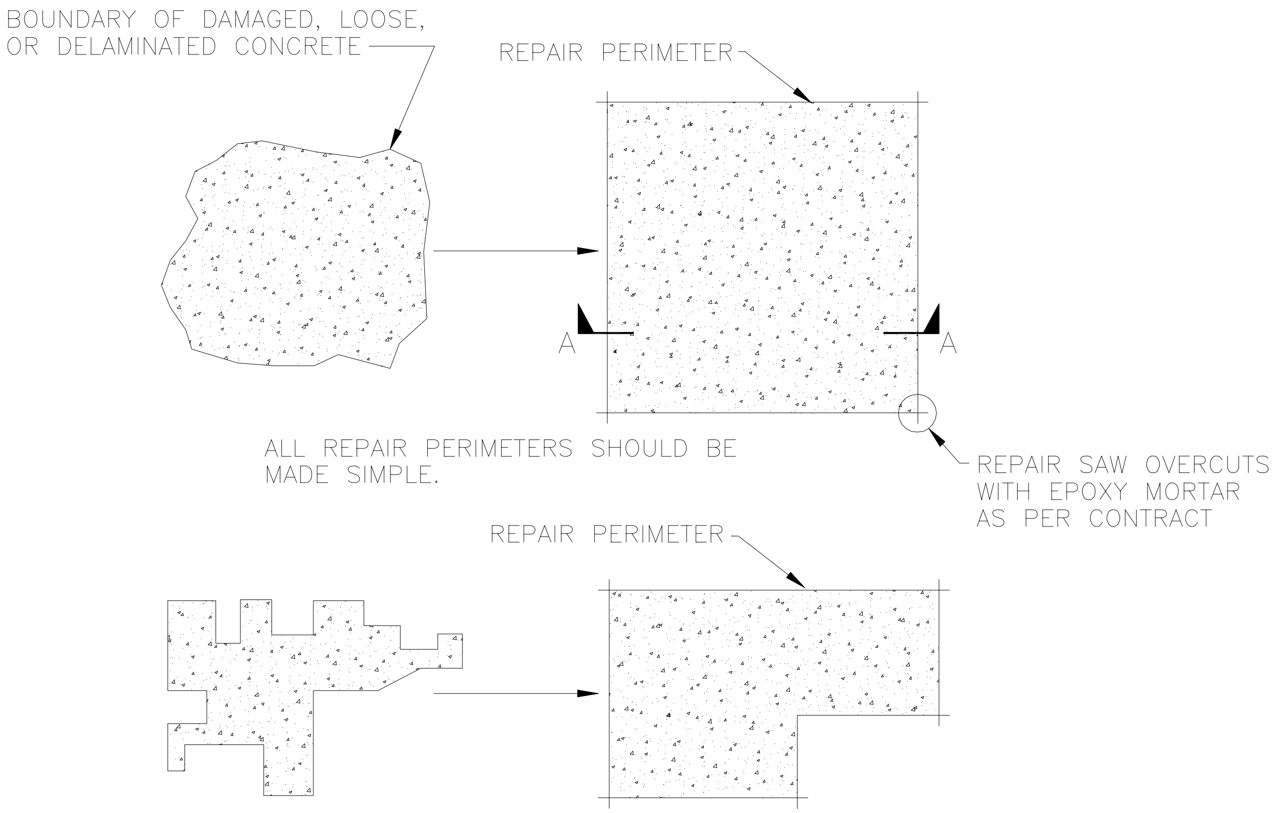
1. ALL CONCRETE REPAIRS ARE OF PARTIAL DEPTH AND SHALL NOT GO THROUGH THE ENTIRE DECK THICKNESS.

2. ALL DEBRIS, ABRASIVE BLASTING GRIT MATERIAL, SAW CUT CEMENT POWDER, AND SLURRY SHALL NOT BE ALLOWED TO ENTER THE WATER.

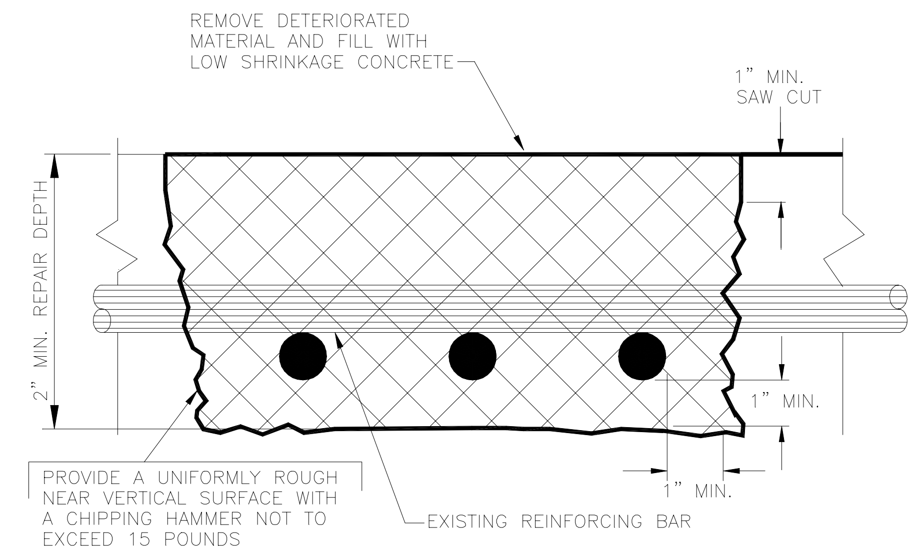
3. LOW SHRINKAGE CONCRETE (OR CEMENTITIOUS PATCH MATERIAL) SHALL HAVE A MAXIMUM ALLOWABLE SHRINKAGE OF 0.05%. (SEE CONTRACT SPECIFICATION FOR MORE REQUIREMENTS.)

4. ALL EXISTING REINFORCING BARS SHALL NOT BE CUT, REMOVED, OR REPLACED WITHOUT THE PRIOR APPROVAL OF THE CONTRACTING OFFICER.

5. USE A REBAR LOCATOR (PACHOMETER) PRIOR TO CUTTING OR REMOVAL OF CONCRETE TO VERIFY DEPTH AND LOCATION OF REBAR. ADJUST DEPTH OF CUT ACCORDINGLY TO AVOID DAMAGING THE REBAR.



TYPICAL REPAIR PERIMETERS

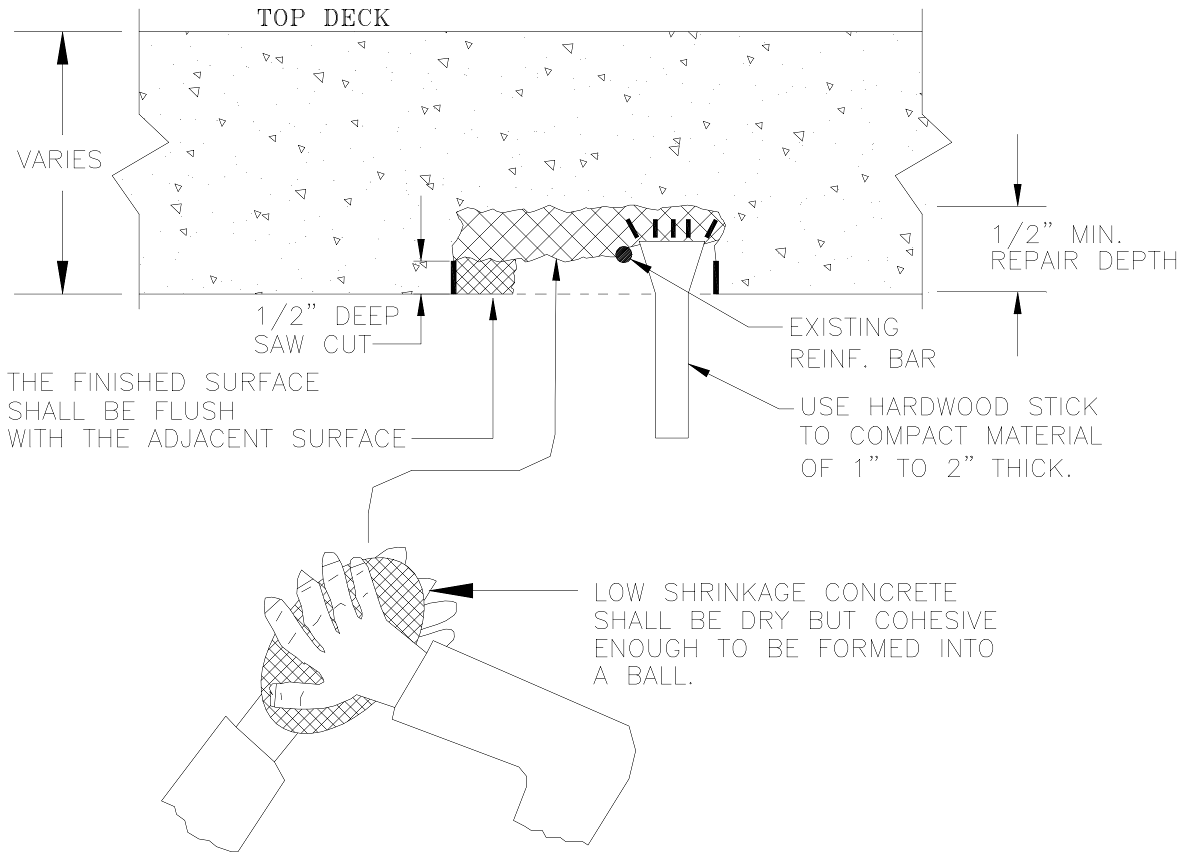


SECTION A–A

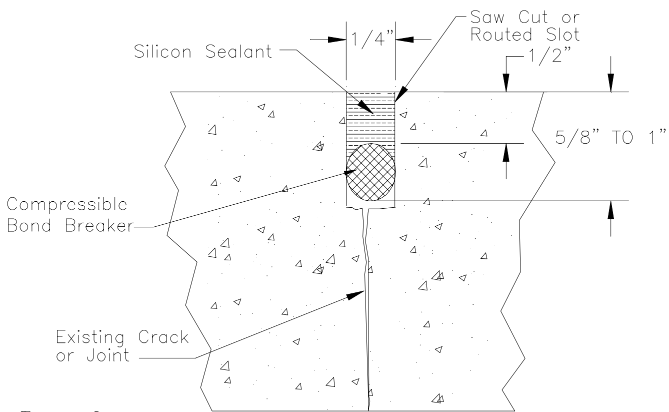
NOTES:

1. SAW CUT (1" DEEP) A REPAIR PERIMETER LAYOUT OF THE DAMAGED AREA.
2. REMOVE DETERIORATED CONCRETE, ABRASIVE BLAST THE STEEL, CLEAN UP ALL CONTAINMENTS & DUST, SATURATE SURFACE WITH WATER FOR 24 HOURS, REMOVE SURFACE WATER, APPLY BONDING AGENT, AND PLACE LOW SHRINKAGE CONCRETE. (SEE CONTRACT SPECIFICATION FOR MORE DETAILS.)
3. CLEAN CORRODED REINFORCING BARS. CUT, REMOVE AND SPLICE WITH NEW REINF. BARS IF THE CROSS SECTION AREA OF THE CLEANED BARS DOES NOT MEET THE MINIMUM REQUIREMENT. SEE CONTRACT SPECIFICATION FOR MORE DETAILS.

PARTIAL DEPTH REPAIRS



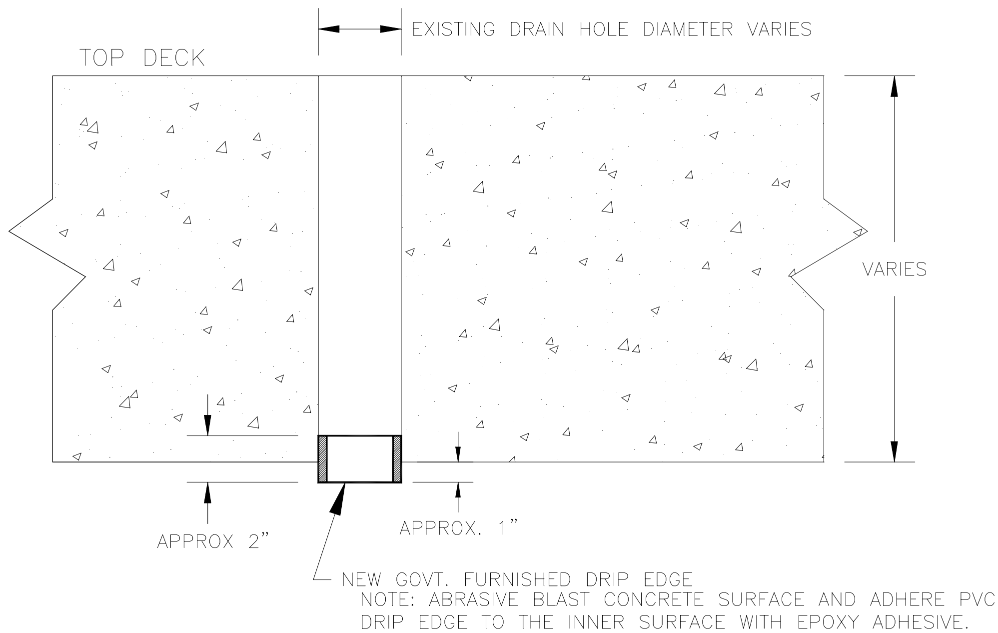
HAND PACK METHOD



Procedure:

1. The Contracting Officer shall mark with chalk the limits of each crack and joint to be sealed.
2. Straight cracks and joints shall be saw cut 1/4" wide by 1" deep and cracks which are not straight shall be routed 1/4" wide by 1" deep.
3. The edges of the opening to receive the sealant shall be abraded to establish an appropriate anchor profile and then air blasted to remove dust and debris.
4. A primer designed to be used with the sealant shall be applied to the prepared concrete edges prior to placement of the bond breaker in accordance with the manufacture's instructions. The Contractor and the Contracting Officer shall inspect the prepared surfaces prior to placement of the bond breaker.
5. A 3/8" diameter compressible bond breaker shall be pressed into the opening at a uniform depth of 1/2".
6. The sealant shall be placed into the opening in accordance with the manufacture's instructions. Care shall be taken to ensure that no voids exist. Excessive sealant shall be removed from the adjacent concrete deck surfaces.
7. The Contractor and the Officer in Charge of Construction shall inspect and probe the cured sealant to assure that it has adhered to the surfaces.

JOINT AND CRACK REPAIR



DRAIN HOLE DRIP EDGE

CHAPTER 9

CASE STUDY: REPAIR OF CORROSION DAMAGED PIER STRUCTURE USING THE FORM AND PRESSURE PUMP METHOD AT PORTSMOUTH, NH NAVAL BASE

CASE STUDY: REPAIR OF CORROSION DAMAGED PIER STRUCTURE USING THE FORM AND PRESSURE PUMP METHOD AT PORTSMOUTH, NH NAVAL BASE

Prepared for a

WORKSHOP

How to Make Today's Repairs Durable for Tomorrow; Corrosion Protection in Concrete Repair

March 21, 1998
Houston, Texas

Prepared by
Douglas Burke
Naval Facilities Engineering Service Center

ABSTRACT

The Navy is continually looking for the best methods to repair its concrete waterfront structures. The goal is to identify methods and materials that provide at least 15 years of service. An evaluation of repairs made to a pier at Portsmouth Navy Shipyard was conducted by the Naval Facilities Engineering Service Center (NFESC). The excellent condition of these repairs after 16 years of service demonstrates that the form and pressure pump method to place a polymer-modified concrete meets the Navy goal. A description of the project, repair methods, and materials is provided.

INTRODUCTION

Many of the Navy waterfront structures are 25 to 75 years of age and require frequent and expensive repairs. Within a Navy pier there are various types of structural elements such as piles, pile caps, and deck slab, each of which may require repair materials and procedures that are appropriate for its specific function, condition, and accessibility.

The typical Navy marine reinforced concrete structure is contaminated with chlorides and rebar corrosion is ongoing. The concrete is often carbonated, resulting in a weakened cement paste at the surface. Conventional repair methods are specified because complete removal of the chloride contamination or the use of cathodic protection methods are often too expensive and therefore excluded from consideration.

CASE STUDY

A case study of the performance of concrete repairs, after 16 years using the form and pump method, was performed in September 1996 at the Portsmouth Naval Shipyard in New Hampshire. This was a major concrete repair project in 1980 involving about 24,000 square feet (2,230 square meters) of repair surface. The general contractor was Peabody NE, Inc. The Navy's engineer was C.J. Foster, Inc., and the subcontractor who performed the repairs was Structural Preservation Systems, Inc. The major part of the project included extensive repairs to the pier's concrete support beams and pile caps.

The beams are exposed to severe conditions in a marine environment. They are located in the splash zone, exposed to wetting and drying, freezing and thawing, and elements of sea water (the Portsmouth harbor has 12-foot (3.7-meter) tides). The existing conditions at the time of the repairs manifested in severe concrete spalling, cracking, and corrosion of embedded steel.

This was the first project where the form and pressure pump method was used. Polymer-modified concrete was used as the repair material. In total, 7,000 cubic feet (198 m^3) of concrete was placed.

The observations during the case study demonstrate that the repairs are in good condition after 16 years in service. It should be noted that in addition to exposure to the severe environment, the pier structures carry heavy cranes, construction equipment, and heavy vehicular traffic. All these loadings cause vibration and impact in the supporting structures.

Several isolated cases of corrosion of embedded steel and concrete deterioration were found in repairs adjacent to steam pipes. Elevated temperatures associated with the steam pipes accelerated the rebar corrosion and concrete deterioration. In general, one can expect that the intrusion of moisture and corrosive agents will be greater at increased ambient temperature and, hence, the time for rebar corrosion will be shorter.

In conversations with the shipyard engineer, he stated that the repairs were very successful and substantially extended the service life of the repaired piers.

The form and pump pressure method was specified and used by the Portsmouth Naval Shipyard in subsequent concrete repair projects performed in 1983 and 1986.

Repair Material

Due to the large size of the project, it was determined that the polymer-modified concrete (PMC) was more cost effective than epoxy mortar. Because the polymer modifier replaces an equal amount of water needed for the concrete, shrinkage cracking is reduced, water permeability is reduced, and freeze-thaw resistance increases.

The mix proportions for the project consisted of 3 parts concrete sand, 1 part portland cement, 2 gallons of epoxy, and approximately 3.5 gallons of water.

Repair Procedure

Deteriorated concrete was removed and the corroded steel was sandblasted to “white” metal.

Form work consisted of 5/8-inch (15.9-mm) and 3/4-inch (19-mm) ply attached to the concrete with expansion anchors. After mixing, the PMC was placed into a positive displacement mortar pump and transported through a 2-inch (51-mm) diameter high pressure hose to the form work. The forms contained a series of ports at 2- to 3-foot (0.6- to 0.9-meter) intervals along the form work. The forms were pressurized to 15 psi.

DESCRIPTION OF THE FORM AND PRESSURE METHOD

The form and pressure pump method is accomplished by placing the repair material into a closed form with a concrete or grout pump. The forms are usually single face forms, enclosing a cavity in a concrete member, vertical or overhead, such as the side or bottom of a beam, a wall cavity, a column cavity, or the bottom of a slab. The main difference between the form and pump method and conventional pumping concrete in a form is the pressurization of the material mix once the form is full. The special form work design allows for the pressurization to be achieved with the power of the pump. This operation significantly improves the bond between the repair material and the existing concrete substrate, and between the repair material and embedded reinforcement. It also allows for a more complete filling of the repair cavity and better consolidation of the repair material than is generally possible with other repair methods. Combined with the use of relatively low shrinkage repair material, this method allows for durable repairs eliminating or minimizing cracking.

Forms

In form and pump (F&P) repairs, forms usually are made of wood materials because they are easy to put together and they are lower in cost than other materials.

As with all forms, F&P objectives in form design are:

> *Quality* - to design and build forms accurately so that the desired size, shape, and finish of the repair are attained while placing quality repair material and providing adequate cover over the steel reinforcement.

Safety - to build form work that will safely support all dead and live loads.

Economy - to build the forms efficiently saving time and money. The less pieces, the more it can be reused.

The main difference between F&P and standard forms is that they must be designed to take not only the full liquid load of the repair material, but also the extra pressure when pressurizing. Design of the forms should follow standard practice for cast-in-place concrete construction, except for the form pressure. Forms should be designed to resist a minimum pressure of 15 psi. They need to be made "tighter" than standard forms, especially for overhead placement; watertight if possible. The forms must be held tight against the existing structure to prevent the new material from wedging between the form and the face of the existing structure.

A sealant such as silicone caulking or urethane foam is used between joints in the plywood and sandwiched between the plywood and the perimeter of the repair area to prevent leakage. Penetrations through the form face are also sealed against leakage.

Placement

In F&P, the repair material is placed into the forms by a pump. The pump must be compatible with the material being placed, and sized to the quantity of material being installed at any one time.

There are many different types of pumps from large truck-mounted boom pumps of 40 to 60 cycles per hour to small moyno types that are rated in gallons per hour. The pump type required depends on the material being placed more than any other factor. Moyno pumps are for mixes that do not contain coarse aggregate (gravel or stone). They are small, easy to move around, and run on 110-volt electrical supply. They are often used on small projects or where the areas to be repaired are small and require a repair material with fine aggregate only.

Hydraulic swing tube pumps, either truck or trailer mounted, are best suited for repair materials with coarse aggregate. The squeeze type pump can also be used.

After the pump is selected, it is necessary to connect it to the form. This is done with either rubber hose or hose and pipe. Pipe offers less resistance to flow of the material than the hose. However, pipe is not flexible, and is difficult to mount onto the form. If there is a long run (several hundred feet between the pump and the form), a combination of pipe and hose is used. Generally, a section of hose is connected from the pump to the steel line and one or two hose sections are then placed on the end of the pipe line to connect to the forms. If there will be several concrete placements in the same area, the steel line is cleaned in place and left for use on the next placement.

Abrupt changes in line size (from larger to smaller) will cause a blockage most of the time. Long tapered reducers shall be used when changing pipe or hose size. For example, if the line size between the pump and the form is 3 or 4 inches and the pump discharge is 5 inches, use a tapered reducer (5 to 4 inches or 3 to 4 inches) between the pump and the line. At the form end of the line, the line size should be reduced further with a tapered reducer to a 2-inch or 2-1/2-inch hose that will be connected directly to the form valve.

For connections between pipe joints or hose sections, it is recommended that a metal clamp with a rubber gasket be used. It is most important that under pressure these connections don’t leak because the mix may become dewatered in the line and cause a blockage. If the connection leaks, it is because of a damaged flange or gasket. Both should be fixed as soon as they are spotted.

It is important to have a gauge in the line where it connects to the form. A recommended gauge size is 200 psi. This gauge capacity is enough to monitor pressure. It is also necessary to have a gauge at the start of the pump discharge line. On hydraulic pumps, an experienced operator can control the line pressure by monitoring the hydraulic system pressure gauge.

Before any pumping starts, a positive, instantaneous communication system between the pump operator and the nozzleman should be established. When the form is full, one stroke of the pump could cause the form to fail if the pump cannot be stopped in time. Do not start pumping until there is direct continuous communication with the pump operator. Two-way radios are best for this purpose.

Precautions should be taken when working with a new pump operator. The operator and nozzleman need to discuss the operation and the signals for directing the pump. The nozzleman should direct the pump operator; the pump operator follows the nozzleman’s directions. Only one person on the crew directs the pump, thereby eliminating confusion and possible injury. The pump operator must be instructed to constantly monitor the radio or the head set. The nozzleman may need to stop the pump at any time. The fewer words used in commands to start or stop the pump, the better. They must be clear and concise and said loud enough for the operator to hear. The importance of clear and constant communication between the pump operator and nozzleman cannot be over emphasized in F&P.

Pump Line Cleaning

After completion of pumping, the line and valves must be cleaned. Be sure to have a proper line sized “go devil” on hand to clean out the line. After the pump pulls as much material from the line as it can, by running in reverse, the “go devil” is inserted in the form end of the line, and an air line attached behind the “go devil.” The remaining concrete is “blown” back to the pump end. Care must be taken to secure the pump line and catch the material including the “go devil” exiting from the line. This procedure can be reversed if it is easier to handle the waste concrete in the line at the form end of the operation.

Pumping

If material is being delivered from a ready mix plant, after the truck is “mixed up” and ready to start discharging, a small amount of the concrete should be discharged into a wheelbarrow to check for proper mixing and slump. Do not put material into the pump hopper before checking it. If cylinders and/or sump tests are to be taken, the material in the wheelbarrow can be used in order to not hold up the placement operation. If the concrete does not appear properly mixed or has an inadequate slump, don’t use it. The cost to remove the material, once it reaches the form, is very high.

After the quality check on the material, slick line (cement mixed with water) is added to the pump hopper and started through the line. This material will “wet out” the inside of the pump line, help seal joints, and prevent the concrete from blocking up in the line. Slick line consists of one or two bags of cement with approximately 5 gallons of water per bag. It is mixed in a mortar mixer, or in buckets, and dumped into the pump hopper. After most of the slick line has been pumped from the hopper through the line, the mixed repair material is placed in the hopper and pumped through the line, behind the slick line.

On the form end of the line, the crew is standing by with a wheelbarrow, buckets, or a 55-gallon drum to catch the slick line material. Do not pump the slick line into the forms, waste it. When the repair material starts to flow from the hose, stop the pump and clean off the connection at the end of the hose line, and connect it to the form.

There should be one or more pumping ports already attached to the form. The hardware should be in place with valves ready for the hose attachment. Start at the bottom or far end of the form, pumping from bottom to top, or from one side to the other, and start with all valves open. As material starts to exit from the valves and/or vents, close the valves and plug the vents. Forms may be pumped from a single port. This is possible if the pressure remains low and the concrete is traveling to all areas of the form. If the pressure starts to rise significantly, the valve shall be closed and the hose disconnected from that pumping port and reconnected to another port where material has already exited. Avoid trapping air between two pumping ports by skipping around. The nozzleman and helpers should have a bucket of water ready to clean off the connections as they change ports. It is necessary to wash the concrete out of the rubber seal and the clamp and grooves in order to reconnect the hose to the pumping ports.

Vibration

For sections 3 to 6 inches deep, vibrate the exterior of the form as the concrete is being pumped in. On deeper placements, as required for Pier 12, it is necessary to install access holes in the form to insert the vibrator. These must be plugged before starting to pressurize the form. Vibration after pressurization may cause unwanted movement or may overstress the forms. Do not vibrate after the forms have been pressurized.

Pressurizing

Pumping will continue until the form is full. At this point, the nozzleman requests the pump operator to give short strokes and monitors the form carefully. He is watching and listening for “cracking” in the form indicating that pressure is straining and/or slightly bulging the form, also an indication that pressure is building inside the form. Once this condition has been achieved, all vents should be capped or plugged off, and all valves closed. The hose can be disconnected from the form.

After the repair material has reached a stage where it will not flow out of the valve (about 20 minutes to 1 hour), unscrew the valves and nipples from the flanges and clean them using a water hose. In most cases the concrete is still green, and a screwdriver and water will remove the material. A wire brush is also helpful to clean the threads. Most of these valves can be taken apart and the ball cleaned and regreased. It is important to take the valves apart and regrease them because the fines will build up and prevent the ball from operating. The valve will now be ready for reuse.

Curing

In the F&P method, the forms must stay in place until the repair gains design strength and becomes self supporting. If the forms are removed too soon, the new material may sag, break bond, and crack. In hot, dry environments, it may be desirable to wet the forms during the curing process to minimize water loss from the concrete and to keep the temperature down. After the forms are removed, the repair should be wet down several times. As soon as the surface becomes dry, immediately apply a curing compound.

CONCLUSIONS

The form and pump method combined with good quality control practices is a good method to repair deteriorated concrete waterfront structures. The expected performance life can exceed 16 years where the ambient temperature is similar to New Hampshire. Performance life for repairs made in warmer climates will be shorter.

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CHAPTER 10

CASE STUDY OF DRYDOCK NO. 6

BACKGROUND

Drydock No. 6 is more than 35 years old. It originally cost about $175 million and the estimated replacement cost would exceed $500 million. Currently, repairs to the concrete are necessary because of corrosion of the steel reinforcement. In 1992, Puget Sound Naval Shipyard (PSNSY) contracted with BERGER/ABAM Engineering, Inc., to prepare construction documents to address the repair of cracks, spalls, and delaminations (Refs 1 through 4). These documents address other repair issues that are not part of the scope of this report. BERGER/ABAM hired Construction Technology Laboratories to conduct a site specific condition survey of the concrete (Ref 5). The shipyard did not award the repair contract as planned, probably due to funding limitations. The original documents are now being updated and revised by BERGER/ABAM Engineers in accordance with Reference 5. PSNSY has requested that NFESC review the project documentation, visit the site, and prepare written recommendations according to a statement of work (Ref 6). BERGER/ABAM is currently updating the condition survey by sounding the walls to the height of 8 feet to obtain a representative sampling of how much the deterioration has progressed in 4 years.

REPAIR OBJECTIVES

Due to the high replacement costs, lack of MCON funding for new construction, and continued mission requirements, PSNSY's objective is to ***extend the performance life of Drydock No. 6 for as long as possible.*** The objectives of the repairs are to:

Minimize the frequency of falling pieces of concrete.
Provide long lasting repairs.
Improve the appearance of the walls.

The scope of work refers to “structural repairs.” Typically, it is very difficult to execute the repair in a manner that provides for the distribution of structural loads through the patched concrete. A structural analysis would likely show that the loss of 6 inches on the surface of the thick cantilever walls is not significant to the performance of the structure.

CORROSION MITIGATION OBJECTIVES

The corrosion mitigation objectives are to:

1. Fill all cracks to minimize penetration of corrosive species to the depth of the steel reinforcement.

2. Apply a penetrating surface sealer to the entire wall area to forestall, retard, or stop any ongoing corrosion processes.

SITE VISIT

The site was visited 6 and 7 January 1997. Joe Sullivan (Ref 7) was the point of contact from the shipyard and Joe Stockwell represented BERGER/ABAM Engineering. The drydock was visually examined and random soundings were made. The Public Works Commanding Officer and Executive Officer were briefed prior to departure. Reportedly, this is the first major repair effort.

The mechanism for deterioration is rebar corrosion due to ingress of salt water to the depth of the steel, especially through vertical control joints and drying shrinkage cracks. About 80 to 90 percent of the wall area appears to be in very good condition. Three large delaminations were indenified as needing to be knocked down immediately to minimize the hazard of falling debris. Standing water was observed in the service gallery. It appears that this water is contributing to the moisture in the vertical cracks and subsequent corrosion of the steel reinforcement. It is recommended that corrective measures be taken.

Prior repairs performed on Drydock No. 3 using pressure injected epoxy and polyurethane grouts were inspected. Both appear to be in excellent condition after 1 to 2 years of service. Concrete repairs using form and pump techniques and machine-applied methods also have performed very well.

HIGH PERFORMANCE REPAIR MATERIALS

Compressive Strength

Compressive strength is typically the defining criterion for the selection of concrete repair materials. In this application, the concrete patch will probably never carry any significant compressive loads. Although cementitious materials have excellent compressive strength characteristics, performance in this case is not a function of compressive strength and need not be a criterion for material selection.

Tensile Strain Capacity

The repair material is subject to drying shrinkage, temperature changes, and flooding the drydock; these will all induce tensile strain. To avoid cracking and delamination of the concrete repair material, one needs to specify a repair in terms of *tensile strain capacity*. Unfortunately, the industry has not yet devised a test which can be correlated to this characteristic. It is known that tensile strain capacity is related, in part, to drying shrinkage, creep, and the modulus of elasticity. A maximum allowable shrinkage and an appropriate test method is recommended.

Some cracking of the repair material will mostly likely occur. Our objective is to minimize the size and frequency of the cracks by using reasonable specifications and qualified inspection to obtain good workmanship and high quality.

STRESSES

Many different loads affect the stresses in the repair material. All materials change volume when subject to stress. The combined stresses on the repair can result in cracking. Cracks relieve the stress but also allow the direct ingress of corrosive species. Cyclic stresses beyond the capacity of the material can result in progress cracking, delamination, and spalling. The combined tensile stresses resulting from drying shrinkage, temperature changes, and flooding need to be resisted by the repair materials.

Drying Shrinkage

Drying shrinkage is a one time event related to the composition of the material, the placement, and curing conditions. As the material cures it shrinks. Because the repair is bonded to the substrate, it is restrained and hence the shrinkage results in tensile stresses in the repair. By curing the repair properly, one can minimize cracks that result from shrinkage stresses.

Temperature Changes

Temperature changes produce cyclic stresses. As these tensional loads are applied to the repaired area, the combined internal tensile stresses may exceed the tensile strength capacity of the repair material.

Operational Flooding

Operational flooding produces cyclic stresses. The concrete repairs are made when the surfaces of the walls are in compression. When the drydock is flooded, the hydrostatic forces will push against the cantilever walls, tending to elongate the exposed surface of the walls. The compressive forces in the *unrepaired wall* will be reduced or reversed. The patched areas will also be elongated, thus inducing tensile stresses in the repair material. These tensile stresses are additive to other stresses.

CRACK INJECTION

Cracks provide direct access for corrosive species to attack the steel reinforcement. Epoxy crack injection can be an effective way to seal stationary cracks. Since epoxy is rigid, it is not effective in sealing cracks that are subject to movement. Cracks move for a number of reasons, such as thermal expansion and contraction, loading, and corrosion of the rebar. It is a common mistake to use epoxy to repair cracks that are caused by rebar corrosion. The expansive steel by-products result in growth of the crack. Epoxy is not sufficiently flexible to accommodate these movements without failure at or near the interface, therefore failures often occur within 1 or 2 years. Most cracks in the drydock are moving or have the potential to move, therefore it is recommended that a flexible sealant be used for the repair of all cracks.

PENETRATING SEALERS

All of the walls of the drydock are to be sealed after the repairs are made, this is intended to slow further chloride penetration and further rebar corrosion. The use of sealers over a new cementitious patch is an effective primary method to retard or to restrict the ingress of corrosive species into the concrete. Sealers are not permanent, and periodic reapplications are necessary. Sealers typically penetrate only a few millimeters. The 1992 condition assessment contains data from several core samples, all of which were taken in cracked or delaminated concrete, except for Core No. 11, which was taken in sound concrete. At this location, chloride ion contamination at 3 inches deep was 1.9 pounds per cubic yard (Ref 5). In the presence of moisture and oxygen, *which is sufficiently available*, the steel rebar is probably corroding. The application of a penetrating surface sealer at the Core No.11 location will slow the ingress of chloride penetration. Given the age and condition of the structure, assuredly, additional areas are contaminated and are experiencing ongoing rebar corrosion, especially along the joints and cracks. Eighty to ninety percent of the surface area appears to be in very good condition and it is recommended that a penetrating sealer be applied over the entire wall to retard the ingress of chlorides.

The frequency of reapplication of the penetrating sealer is uncertain. It is recommended that the chloride ion contamination be measured annually at three specific locations at 1 inch, 2 inches, and 3 inches deep so as to monitor and document the level of contamination. By doing so, the command will have data to help make decisions on the frequency of reapplication and its effectiveness.

PERFORMANCE EXPECTATIONS

The performance life of a repair is extremely difficult to predict, as the performance life depends on the material selection, method of application, and quality of the workmanship. The ROICC should strive to obtain the highest quality workmanship from the contractor. Continuous inspection of the repair process is a method to help maintain uniform and continuous high standards throughout the project. The need for ***continous qualified inspection*** of the repair contract can not be over emphasized. If all of these items are correctly accomplished, the repaired areas should function for 20 years.

Outside the repair areas the interior and external environmental conditions can produce new delaminations. It is common to see new delaminations occur adjacent to patches in 1 to 3 years. There are two dominate reasons why this occurs. They are:

> The extent of deterioration is often underestimated. Because the *area must be chipped back to sound concrete and to uncorroded rebar,* it is impossible to foresee the repair boundaries in a non-destructive condition assessment. Good inspection can minimize this problem.

Rebar corrosion is accelerated by differences in conditions along a continuous rebar. After the high quality repairs are completed, a portion of the rebar is protected by the new high quality patch, while the steel extending into the adjacent concrete is exposed to relatively severe conditions. These differences promote a strong corrosion cell resulting in accelerated rebar corrosion and possible delamination of the concrete near the patch and occasionally at the patch itself. Life extension methods can address this problem.

LIFE EXTENSION METHODS

The use of impressed current cathodic protection (ICCP) can be used to arrest the corrosion process. This method can provide a 15- to 20-year solution. However, there are many issues that must be addressed and considered in the design, installation, and operation of an ICCP system. The application of the ICCP system requires several protection zones, each with electrical supply and control circuits. The external application of a flame sprayed titanium distribution anode may not be sufficiently durable to resist the wear and tear on the lower portion of the wall. It is possible to cover the anode with epoxy or to cut grooves and embed the anode.

In 1993, NFESC prepared a comprehensive technical assessment of the technology (Ref 8). Currently, NFESC is performing cooperative research with industry to resolve installation and performance issues. In FY97, NFESC plans to install a 1,000-square-foot demonstration of an ICCP system on the substructure of a Navy pier at SUBASE San Diego. The system performance will be monitored for at least 1 year. Results after 1 year of operation will provide valuable installation and performance data necessary for the transition of this technology to Navy marine structures. It is recommended that the use of this technology for Drydock No. 6 be postponed until these tests are completed.

Chloride ion removal techniques may be a viable candidate for future consideration. An assessment of the feasibility of employing this emerging technology is worthy of consideration. NFESC will attempt to include this investigation into the current scope of work funded by Naval Facilities Engineering Command Headquarters (NAVFACHQ).

ECONOMY OF ICCP AND CHLORIDE REMOVAL

Currently, PSNSY needs to spend at least $6 million to make repairs. With a rough estimate at $20 per square foot, $3 million would be required to install an ICCP system to stop the corrosion cell. The life expectancy of this system is 20 years and would eliminate the need to spend $6 million every 5 to 10 years to repair the concrete due to rebar corrosion. Electrical continuity must be determined. Typically, structures contain sufficient continuity, but one must confirm it. If technically feasible, the benefits-to-cost ratio of chloride ion removal will probably be comparable to using ICCP.

REPAIR TECHNIQUES AND MATERIALS

The recommended repair material and application method is based upon the following criteria (this list was developed from experience on other Navy jobs and discussions with industry and Navy experts):

The operational costs associated with repairing a drydock dictate that, within reason, the very best repair technique and materials should be selected.

The ultimate long-term properties of the repair material are far more important than the ease of application.

Bond of the repair material to the concrete substrate depends mostly on mechanical interlocking.

There are no repair techniques or materials that are tolerant to the applicator's lack of experience, workmanship, and quality control.

Segregation of the repair material will alter the repair material's physical properties.

The greatest problem in concrete repair is to minimize the amount of cracking. A network of micro-cracks and visible cracks provides transport for corrosive agents to the rebar.

When selecting the repair method, it is best to specify only one and at most two methods for the same project.

The use of *machine applied material (shotcrete*) is especially sensitive to applicator experience, materials and equipment, workmanship, and quality control. Generally, its use is discouraged. Improper surface preparation is largely responsible for failure of the repair. Large cumulative areas of repairs, less than 3 inches thick and containing very little rebar, may be most efficiently repaired using shotcrete, although *hand application* methods typically perform better.

Vertical repairs containing closely spaced rebar and more than 20 square feet and at least 3 inches deep are best performed using the *form and pump* method.

Hand-applied dry packing is best for small areas.

REPAIR MATERIALS

It is recommended that a prepackaged repair material be used to achieve the low shrinkage required. Shrinkage is the most important property. Because of product variability it is recommended that a third party independent testing laboratory verify that the material delivered to the job site meets the criteria.

Drying shrinkage shall not exceed 0.05 percent at 28 days per ASTM C 157 modified to use molds per ASTM C490 (3 x 3 x 11.25 inches) with a 10-inch gauge length. During the first 24 hours of curing, the molded specimen shall be cured at 46 to 54 percent relative humidity at 70 to 76 F. After 24 hours, remove the mold and cure as prescribed in the standard.

The following prepackaged products have been used and tested for other repair projects and have properties consistent with the objectives of this project:

Manufacturer	Phone	Product Name
Five Star	202-336-7900	Five Star Structural Concrete V/O
Master Builders	216-831-5500	Emaco S66-CR
Fosroc	800-441-3633	Renderoc LA
Sika	800-933-7452	Sika Top 111 Plus
Euclid	216-531-9222	Euco SR-93

DRY PACKING AND HAND-APPLIED METHODS

Dry packing is the recommended technique to repair small cavities. Hand placement can also be an acceptable method. Both application methods are similar and the contractor should follow the manufacturer's recommendations. In both methods, it is very important to have good consolidation in and around the rebar and between the layers. A vertical shoulder is necessary to provide lateral restraint when packing or placing the material. A shoulder will be established at the perimeter by cutting the concrete with a saw and chipping the interior. The material shall meet the shrinkage criteria of 0.05 percent.

Dry packing is a repair method of placing zero-slump, or near zero-slump, concrete or mortar by ramming it into surface cavities. Sufficient water should be used to produce a mix that will stick together while being molded into a ball with the hands and that will not exude water but will leave the hands damp. Less water will not make a sound, solid pack, and may result in excessive shrinkage and failure.

Hand-applied techniques use a non-sag material often with special blends of cement. Skill of the applicator is important to obtain good consolidation and bond to the substrate.

A bonding agent consisting of either neat cement, cement-sand, or latex-cement-sand slurry shall be used because dry pack lacks the extra moisture necessary to promote good bond. Compaction densifies the repair material and provides the necessary contact with the existing concrete for achieving adequate bond. The action of the trowel is used in hand placement to accomplish sound consolidation. In the dry pack method, a hardwood stick and a hammer is used. These sticks are usually 8 to 12 inches long, and are used in preference to metal bars because the latter tend to polish the surface of each layer and thus make bond less certain and repair less uniform. Much of the tamping should be directed at a slight angle and toward the sides of the cavity to assure maximum compaction in these areas. Dry pack and hand placement are usually placed in layers depending on the repair thickness.

Because of the relatively small volume of most repairs and the tendency of old concrete to absorb moisture from new material, water curing is necessary for the first 72 hours followed immediately by a sprayed curing compound.

FORM AND PUMP

The form and pump method is the recommended method to place concrete repair material. The conventional application of this method has a shortcoming in that the repair material typically does not completely fill to the top of the form, leaving a small gap at the top. This crack must be filled. Options are to inject the crack or to hand pack a cementitious mortar into a routed out joint. Alternatively, the repair material can be placed using the pressure pump method, which fills the form more completely. Another variation is to prepack the form with a graded aggregate and then pump a very high slump cementitious slurry. NFESC recommended this technique in June 1987 at Camp Courtney, Okinawa, Japan with success (Ref 9).

FORM AND PRESSURE PUMP

The form and pressure pump method was first used by the Navy at Portsmouth Naval Shipyard in 1983 and 1986. A site inspection performed in September 1996 showed these repairs to be in excellent condition. The repair material is placed into a closed form with a concrete or grout pump. The forms are usually single faced and pressurization is achieved with the power of the pump. This operation significantly improves the bond between the repair and the substrate and between the repair material and the embedded steel reinforcement. It also allows for a more complete filling of the repair cavity and better consolidation than conventional methods**.** ***Use of a very low shrinkage repair material with the form and pressure pump application method is considered the best alternative and therefore it is the recommended method.***

SURFACE PREPARATION AND CONDITIONING

Surface preparation and conditioning is a critical phase of the repair process. ***Continuous and knowledgeable inspection*** is recommended to assure high quality surface preparation.

The limits of the repair areas should be marked and a decision made on how to "square up" or combine the adjacent areas to simplify the repair geometry and reduce boundary edge length. Excessive or complex edge conditions result in shrinkage stress concentrations and cracking

Most of the removal work is done by small hand-held chipping hammers because of the mobility and versatility these tools allow. In addition, they do the least amount of damage to the remaining concrete and reinforcement. Impact hammers in the 15-pound class are recommended. Impact tools greater than 15-pound hammers can cause cracks in the sound concrete and should be strictly prohibited. The chipping hammer provides a very rough surface texture which improves aggregate interlock at the bonding surface.

The inspector and contractor should be instructed to start chipping a few inches away from the boundary lines. After the rebar is exposed it may be necessary to modify the boundary layout. Rebar with visible corrosion must be "chased" with the chipping hammer to the point where no surface corrosion is visible. Care must be taken to not damage the rebar. It then may be necessary to redefine the boundary of the repair area. Saw cut the perimeter at 90 degrees at least 1 inch deep. The depth of removal is determined by the depth of unsound concrete, but must be at least 1 inch behind the rebar. When completed, the surface shall be clean, sound, and with uniform roughness. After the forms are constructed and prior to placement, the surface should be "Saturated Surface Dry" (SSD). Never pump repair material into a form containing standing water.

FORMS

Forms are typically custom made of wood. They must be designed to resist the full liquid load of the repair material plus 15 psi from pressurization of the repair when using the *form and pressure pump method.* The forms need to be secured tightly to the wall to prevent the repair material from wedging between the form and the face of the wall. Expansion anchors are typically used. The boundary needs to be sealed with urethane or silicone caulking to prevent leakage of the liquid cement mortar. Excessive loss of liquid cement mortar results in honeycombing of the repair material, allowing rapid ingress of salt water to the rebar.

PLACEMENT

Placement of the material is by pump. The pump must be sized to be compatible with the materials and size of the repair. Many types of pumps are available. For repair sections greater than 3 inches thick, vibrate the exterior of the form as the concrete is being pumped. Pumping will continue until the form is full. A pressure gauge installed on the supply hose near the form is monitored carefully. Care should be exercised to observe and listen to the forms for distinctive bulging and cracking. Vents are capped off and the supply valve is closed. Do not vibrate the forms after pressurization. Remove the supply hose. Allow the material to take an initial set, about 30 to 60 minutes, then remove the valves and nipples for cleanup. Leave the forms in place for as long as practical, a minimum of 48 hours. Immediately apply a curing compound to the surface.

SITE VISIT - May 1998 Concrete Repairs Delivery Orders 007 and 0083

Objective

The objective of this section is to document the findings and recommendations of the site visit performed on 21 May 1998 at Puget Sound Naval Shipyard (PSNSY), Bremerton Washington, Drydock No. 6.

Scope

The scope of this effort includes:

1. Conduct a site inspection.
2. Recommend testing to access the effectiveness of the concrete repairs.
3. Recommend any rework.
4. Identify any "lessons learned" to be incorporated into the remaining work.
5. Provide an outbrief to PSNSY personnel.
6. Provide a written summary of findings and recommendations.

Contract Documents

The project specifications are the link between the owner's vision and the construction of the project. They provide a written vehicle between the Navy and the contractor to meet the Navy's needs. To accomplish this goal, the specifications must be easy to understand and to implement, therefore, good specifications are fundamental to the projec's success. Oversimplification or ambiguity of specifications can lead to confusion, overbidding by the contractor, and poor quality.

The repair strategy was to use a method known as form and pressure pump. This method was pioneered for the Navy at Portsmouth Naval Shipyard in 1980. The Naval Facilities Engineering Service Center (NFESC) inspected the form and pressure pump repairs at Portsmouth in 1996 and found them to be in excellent condition.

Crack repair procedures were modeled from successful crack injection used on Drydock No. 3 PSNSY in 1994. These repairs have performed very well.

The contract documents are complete and clearly written. They delineate state-of-the-art concrete repair procedures using conventional methods and materials. The repair methods specified have a track history of being constructable and durable.

Site Inspection

Access to the wall repairs was limited to the areas from the drydock floor to an elevation of about +7 feet. In addition, the repairs in the galleries were inspected. The repairs include four types:

1. Wall repairs using the form and pressure pump method with Renderoc LA.
2. Surface repairs using the hand-applied method with Fosroc SP25 for small areas.

3. Crack injection using high pressure and mechanical packers using WEBEC 1403.
4. Application of a penetrating sealer.

Form and Pressure Pump Wall Repairs

The material used for the wall repair was Fosroc Renderoc LA. No data was found in the contract files to document that this material passed the required tests for shrinkage per American Society for Testing and Materials (ASTM) A157 modified. Independent tests by NFESC, not related to this investigation, indicated that this material has a very low shrinkage rate of 0.02 percent, which is less than the criteria of 0.05 percent. The surfaces are cracked more than expected.

There are several reasons why cracks occur in a concrete repair including: plastic shrinkage, drying shrinkage, reflective cracking, changes in temperature and humidity, disbondment, incorrect placement, improper joints, and inadequate curing. Although the cracks in these repairs are more frequent than expected by the Navy, they are within the allowable tolerances for shrinkage and are consistent with typical construction practices. Moisture was apparent at the surface of many of the cracks and that is an indicator that the repairs may not last as long as the Navy expected.

Generally, the form and pump repairs appear to be sound, they “ring” when tapped with a hammer. Most of the repairs have a random-shaped perimeter. Core locations were identified to obtain samples to evaluate the bond and consolidation of the repair material.

Core R1 was taken from the northeast wall, station 15+00E elevation 80 feet. This core, like the others taken for this investigation, was 4 inches in diameter and about 10 inches long. The repair material in core R1 was about 6 inches deep. Visual examination of core R1 shows a sound well-consolidated repair that is securely bonded to the concrete substrate. Core R2 was taken from the northeast wall, station 15+10E elevation 97 feet. Core R2 was well consolidated and broke in half near the interface between the repair and the substrate and the bond appears adequate. Core R3 was drilled from the northeast wall, station 14 +40 elevation 77 feet and was well consolidated and bonded to the substrate. In addition, the core broke in half about 3 inches deeper than the repair interface. At this location, a large lens of sand and water was present.

Most of the form and pump repairs appear to have <u>not</u> been filled correctly. Reportedly, the forms were not sufficiently strong and consequently the forms bulged and in some cases blew out. Evidence of cement adhered to the wall below the repairs seems to confirm this. The projected surface of the repair material from the original surface profile is acceptable from a durability and performance point of view. The abrupt edges were ground to transition with the adjacent surfaces. These cut surfaces contained very high amounts of entrapped air which is likely to be very permeable to the ingress of chlorides, water, oxygen, and carbon dioxide; all of which will shorten the life of the repair.

The technique used by the contractor to prepare the perimeter was inconsistent. The drawings require a 1-inch vertical shoulder at the perimeter. It appears that the perimeter was not always cut at 90 degrees to the surface but was chamfered at up to 45 degrees and filled with a fillet of concrete. Feather repairs and edges cut at 45 degrees are susceptible to spalling and several repairs in the southwest corner at about station 5 have already disbonded.

The forms were not filled completely and often a void was left to be filled in multiple lifts. Reportedly, the forms were not consistently vented which prevented the material from filling the forms completely. The void at the top of the form was filled by hand placement with Fosroc SP25. This should not have occurred and, as a result, vastly increased the quantities of hand-placed materials. Generally, materials placed by hand will be of lesser quality than materials placed by the form and pressure pump method. Therefore, this defect will manifest as poorer durability for the Navy.

Small Area Repairs by the Hand-Applied Method

The hand-applied method was specified for areas less than 2 square feet. The material selected for these repairs was Fosroc SP25. In addition, it was used for the areas not properly filled by the form and pump method. No documentation of the shrinkage characteristics of Fosroc SP25 was found in the contract file. Many of the repairs performed with Fosroc SP25 were tapped with a hammer and sounded "dull." Their integrity and bond to the substrate of all of the SP25 repairs are questionable. Fosroc was contacted to inquire about the shrinkage properties of Fosroc SP25 and they reported that the product had been discontinued. One should question if the product used on Drydock No. 6 was fresh and within acceptable shelf life. Shrinkage data from Fosroc, using ASTM 157, unmodified, indicates that the material will likely not conform to the job specifications. Core R4 was taken through the SP25 repair. The thickness of the repair in this core was about 4 inches. The SP25 was completely saturated with water and unbonded to the substrate. The repair material acts as if it were a sponge, soaking up water. In addition, the core contained a small bolt of unknown origin. It is recommended that all repairs done with this product be removed and reworked correctly.

Crack Injection

Procedures for proper crack injection require that cracks greater than 0.020 inches wide must be sealed with an epoxy crack sealer (03931 section 3.1.1 of the specifications) and smaller cracks may also require sealing. "Before a crack can be injected, it must be sealed at the surface *(with a cap)* to prevent the resin from escaping. No problem has frustrated the crack injection process as much as cap leaking. If the cap is not installed properly, the consequences are costly. For the cap to bond properly to the concrete, the surface must be sound, clean and dry. The cap must be rigid to keep injection pressures from causing it to quickly blister and rupture or slowly peel away. A thick cap, 1/8-inch minimum, 3/16-inch optimum, will stay rigid. A high-modulus, 100% solids, moisture-tolerant epoxy is often the resin of choice for capping. ASTM C 881, Types I and IV, are usually most appropriate." [1]

Core C1 contained resin in a delamination. The resin did not appear to be bonded adequately to the concrete. An injection porthole of about 1/2-inch diameter was intersected by the core, it should have been filled with urethane or cementitious material but was not.

Core C2 contained an injection porthole that was not filled with resin or concrete. The core also contained a delamination that had some resin in it that appeared not to be adequately bonded to the concrete.

[1] Trout, John, Epoxy Injection in Construction, The Aberdeen Group 1997

Core C3A was taken through a crack with a cap. The cap was very thin, about 0.01-inch thick, and therefore not sufficiently rigid. The crack had no resin in it.

Core 5 was taken through a crack which was about 0.02-inch wide. There was no cap over the crack. The crack was not completely filled with resin and the bond of the resin to the concrete appeared inadequate. There was a delamination at a depth of about 4-inches and there was some resin in the delamination which was not well adhered.

In summary, pressure crack injection of the walls appeared to be grossly inadequate. Surface preparation prior to application of the cap appeared to be entirely inadequate. None of the cracks appeared to have been sealed correctly with a rigid cap. None of the injected cracks inspected by core examinations were satisfactorily repaired. It is recommended that the injected cracks be completely (100%) reworked, unless that contractor can demonstrate on a case-by-case examination that the crack has been repaired correctly.

Penetrating Surface Sealer

For penetrating surface sealers to seal the wall from the intrusion of salt water, they must be applied to a clean surface. This essential aspect of the work was clearly stated in 07180 section 3.1.2). No attempt was made by the contractor to clean the surface prior to application of the sealer. The contractor should have scraped all of the cement slurry, cleaned off contaminates, and water blasted. In addition, no documentation was discovered in the contract file to verify that the quality assurance requirements stated in 07189 section 1.8 were accomplished. No tests were performed to detect the presence of the sealer. The effectiveness of the sealer as applied, if applied, is probably nearly worthless because the surface was not properly prepared. This work should be redone properly.

Recommendations for Rework

Wall Repairs. Identify edges that are likely to disbond and repair them according to the specifications with a 1-inch vertical shoulder. Never permit fillet repairs and featheredges. Remove all repairs done with Fosroc SP25 and rework correctly.

Cracks. Rework all (100%) of the cracks to assure that the cracks are completely sealed.

Surface Sealer. Remove debris, clean and apply the sealer properly.

Recommendations for Remaining Repairs

Preparation. A vertical shoulder at the perimeter of the repair is necessary to prevent spalling of the repair at the edges. This practice was not always followed, it is important to do so in all future work to avoid spalling at the boundary.

Placement. Good construction practices for using the form and pressure pump method are contained in NFESC "Concrete Repair Recommendations and Specifications," by Douglas F. Burke, April 1997. Future work should follow these guidelines. A minimum number of cold joints between vertical lifts are desirable, 10-foot lifts should be a minimum goal. The forms must be filled completely and under 15-psi pressure to assure maximum bond and durability.

Cracks. Existing cracks that coincide with wall repairs will reflect through the bonded repair. These cracks must be injected prior to the concrete repairs.

Quality Control. The literature recommends that strict adherence to quality control is necessary to assure that good concrete placement practices are followed. The Job Order Contract must contain necessary quality control requirements to assure that the repairs are accomplished satisfactorily. Work to date indicates a lack of attention to quality assurance issues related to long performance and durability of the repairs. A review of these procedures is recommended. A meeting is recommended to discuss the procedures prior to continuation of the repair work. Prior to continuation of the work on the head wall, the contractor shall successfully demonstrate to the Navy a successful method of forming, placing, and curing Renderoc LA using the pressure pump method. In addition, methods for crack injection should also be demonstrated. Continuation of the project should not proceed until all parties have witnessed and approved the procedures. Completion of the demonstration repair will set the standard for all future work.

Sealers. Penetrating sealers must be applied after the surface is properly cleaned and the technical representative has trained the contractor.

Commentary

Repairs. About half of all concrete repairs fail in the first few years and 99 percent of those fail due to either the use of inappropriate repair materials or inadequate surface preparation, or both. Cracks and disbondment are usually progressive and will continue until the function of the facility becomes impaired or unsuitable for its intended purpose. The hydrostatic head behind the walls will force water through any cracks not repaired in the wall. Water is very detrimental to concrete durability.

Crack Injection. Injection of the drydock is likely to require extremely high quantities (cubic feet) of resin to fill the cracks and the myriad of unknown intersecting voids in the concrete wall. When the surface of the crack is sealed, the resin is forced into the cracks and intersecting voids until all of the spaces are completely filled and enough back pressure develops to force the material out of the next injection port. If the crack goes completely through the wall, then the resin may flow into the backfill. In the galleries both sides of the crack must be sealed prior to injection. The contractor should add an accelerator to the resin so that it sets prior to flowing out the backside. The amount of pressure used during injection is also critical and directly affects the amount of resin that will be used. This aspect of the project is most difficult to administer because the contractor is being paid by the linear footage and not by total volume of resin used. Consequently, a burden is placed on both the contractor and the Navy to work out an acceptable procedure, depth of penetration, and method of payment.

SITE VISIT - October 1998 Drydock No. 6 Repairs and Inspection at PSNSY

Problem

The Shipyard is concerned about the inconsistent quality of the concrete repairs and "excessive cracks" in the repairs recently completed on the headwall, phase 3 work order 0157, Drydock No. 6.

Approach

The Shipyard invited many individuals including the material supplier to visit the site, observe the work completed, observe a demonstration placement, and to participate in discussions. Douglas Burke of NFESC attended on October 22-23, 1998.

Objective

The Shipyard's objective is the have the contractor (Del-Jen) and the material manufacturer (Fosroc) discuss modifications to the methods and materials that will result in future repairs that have fewer cracks.

Headwall Inspection

The repairs at the headwall are complete. They are large and vary in size from 30 to 217 inches wide by 33 to 215 inches high by 6.5 to 10 inches thick. During a prior visit in August 1998, it was observed that the deteriorated concrete had been removed completely around the existing reinforcement, which is at about 12 inches o.c. with about 5 inches of cover to the surface. The cementitious repair material used was a prebagged mortar mix was Fosroc Rederoc LA. The concrete mortar was placed by the form and pump method about 30 days before this inspection. The forms were reportedly removed between 4 and 7 days after concrete placement, some of the cracks were reportedly visible at that time. The dock was then flooded and subsequently dewatered. On October 10, when the repaired areas were still wet, the cracks were photographed. The photos clearly show many cracks. No photos were made available for this report. The crack pattern is representative of differential drying shrinkage.

Cracks were measured on the headwall. Only the major cracks were visible on the day of inspection because the wall was dry and dusty. In contrast, the photographs show many other cracks that are narrower in width. The major cracks are about 12 to 20 inches apart in both directions. About a dozen cracks in various repairs were measured, their width varied from 0.002 to 0.005 inches. Acceptable industry standards permit 0.05 percent shrinkage at 30 days. The permissible crack width over a representative 16-inch crack-to-crack spacing is (0.0005) (16 inches)= 0.008 inches. This maximum allowable crack width is greater than any of the cracks measured. On average, the cracks in the headwall are about 0.02 percent at 30 days, an acceptable value.

Laboratory Shrinkage Data

Shrinkage tests conducted in the laboratory indicate that this specific repair product will shrink 0.013 percent in 30 days at 50 percent relative humidity. These values compare favorably to the measurements taken on site.

Placement Demonstration

During the demonstration the following important observations were made:

The water monitoring tube on the mixer was new and marked with duct tape at a level that corresponded to 3.5 quarts of water per 55 pounds of dry mortar.

A discarded water monitoring tube was discovered adjacent to the mixing machine. Duct tape had been used to mark a setting that was significantly higher than marked on the new tube. The old tube was caked with mortar and no longer transparent.

The concrete substrate had not been flooded with water to saturate the substrate per the contractor's written procedures.

The forms were not watertight.

The mortar had no coarse aggregate in it.

No vibration was used on the forms to consolidate the fresh concrete and apparently none is required because the consistence of the mixture is extremely fluid.

The delivery hose used to supply fresh mortar from the pump to the form appeared to have a nominal 1.5-inch diameter and was about 200 to 300 feet long. This size may be too small to allow reliable delivery of the mortar from the pump to the form without clogging.

A fresh concrete mixture of 3.5 quarts per 55 pounds of dry mortar was successfully pumped through the 1.5-inch hose to the form.

When the form was filled up 5 feet, an experiment was conducted to slightly reduce the water content by an unknown amount. The purpose was to allow for an evaluation of the hardened concrete to determine if there were fewer cracks associated with the reduced water content. However, soon after the water was reduced the hose clogged and the experiment terminated.

No slump measurements were made and apparently there are no quality control procedures established to document that consistent and high quality concrete is being mixed and delivered to the forms.

Performance Exceptions

Cracks permit the premature ingress of salt water that will ultimately result in the deterioration of the concrete and the steel reinforcement. All of the cracks will continue to increase in width as the repair material continues to hydrate over the next year. Consequently, every effort should be made to reduce crack frequency and width on future repairs.

Differential Drying Shrinkage Cracks

In general, the degree to which differential drying shrinkage and associated cracking can be minimized is improved by using a concrete mixture that contains the correct gradation of aggregates, and a size of coarse aggregate appropriate for the thickness of the repair and the placement method. The repair area would shrink less if it contained a coarse aggregate. The Renderoc LA does not contain coarse aggregate. Repairs greater than 4 inches thick should use a well graded 1-inch minus aggregate conforming to ASTM C33. Ready mix concrete is an acceptable substitute to the prebagged material used.

The substrate surface should be completely saturated with clean water at least 24 hours prior to placement and then allowed to surface dry (saturated surface dry).

Ensure that all of the forms remain in place for a minimum of 7 days. In addition, in hot conditions it is desirable to keep the forms wet during the entire 7-day curing period to minimize the water loss from the concrete. Apply two coats of a curing compound immediately after form removal.

Conclusions

The cracks in the headwall repairs are due to differential drying shrinkage. They will ultimately have an adverse effect on the life expectancy of the repairs and the drydock. The crack frequency and widths are within the manufacturer's and contract specifications. However, it is feasible for the contractor in collaboration with the material manufacturer to produce future repairs that contain fewer cracks. The group discussions and placement demonstration should provide the contractor and the material representative with sufficient information to allow them to formulate a procedure for future repair work that will result in repairs that contain fewer and smaller cracks.

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CHAPTER 11

UNDERWATER CONCRETE INSPECTION AND REPAIRS

The following section contains excerpts from NAVFAC P-990 for underwater concrete inspection and repairs. This document is available through Stanley Black, NFESC, Phone: 805-982-1002; e-mail blacksa@nfesc.navy.mil.